# HUMBER SEWERSHED

# COMBINED SEWER OVERFLOW STUDY

TECHNICAL REPORT # 7

A REPORT
OF THE

TORONTO AREA WATERSHED
MANAGEMENT STRATEGY
STEERING COMMITTEE

June, 1986

# TORONTO AREA WATERSHED MANAGEMENT STRATEGY STEERING COMMITTEE MEMBER AGENCIES

Ontario Ministry of the Environment

Metropolitan Toronto and Region Conservation Authority

Municipality of Metropolitan Toronto

Borough of East York

City of Etobicoke

City of North York

City of Scarborough

City of Toronto

City of York

Regional Municipality of Peel

Regional Municipality of York

Environment Canada

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STEERING COMMITTEE


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A study of this nature and scale is necessarily a team effort. The author's many thanks are due to all the participants and supporters involved.

## SUMMARY AND CONCLUSIONS

### 1.0 Study Objective

The objective of the study is to provide the Toronto Area Watershed Management Strategy Study (TAWMS) with information concerning combined sewer overflows (CSO) to the Humber river and possible control of the CSO pollution. TAWMS plans to use the information for the development of a comprehensive water quality management strategy for the Humber river.

### 2.0 Study Area Characteristics

2.1 The study area comprises a combined sewer area and a sanitary sewer area in Metro Toronto in the Humber river watershed. The two areas share the same Metro sewer and the Humber Water Pollution Control Plant (WPCP) for conveyance and treatment of their sewage. By including the sanitary sewer area in the study, the total hydraulic loading on the Metro facilities can be determined in the case of CSO control measures affecting these facilities.

The following are basic data of the two sewer areas under existing (1983) conditions:

| | Combined Sewer Area | Sanitary Sewer Area |
|---|---|---|
| Area | 1,246 ha | 17,418 ha |
| Population | 81,000 | 449,000 |
| Dry-weather flow | 36,000 $m^3/d$ | 320,000 $m^3/d$ |

The combined sewer area is 7% of the study area.

2.2 In wet weather, combined sewage up to 2.43 $m^3/s$ (5.8 times of dry-weather flow of the combined sewer area) is allowed to flow to the WPCP. Flow in excess of this limit is partly diverted to an existing detention tank (Hyde Avenue tank) and the remainder

overflows to the Black Creek (a tributary to the Humber river) via 3 regulators set in the sewers. The tank has a capacity of 7,800 $m^3$. When full, the tank also overflows to the Black Creek. The detained flow is returned to the WPCP after a storm.

2.3 Monitoring results in April-October, 1983 indicate that the regulators overflow to the Black Creek if the storm has more than 4 mm precipitation. The smallest storm in which the existing tank is filled has 7.4 mm precipitation.

The observed, flow-weighted average pollutant concentrations of the combined sewage are as follows:

| | mg/l | | mg/ l |
|---|---|---|---|
| Suspended solids | 196 | $BOD_5$ | 55 |
| Total phosphorus | 1.96 | Soluble phosphorus | 0.44 |
| Lead | 0.182 | Zinc | 0.300 |
| Copper | 0.119 | Cadmium | 0.006 |

Fecal coliforms (counts/100 ml) $1.65 \times 10^6$
The results do not show any abnormal concentrations.

2.4 In addition to the above-mentioned regulators, an overflow regulator (near Berry Road) exists in the Metro trunk sewer just upstream of the WPCP. Monitoring results of this regulator in June-October, 1984 show that it overflows sparingly in a thunderstorm only and the overflow duration is about 1 hour. This observation supports the model simulation results in paragraph 4.2 that this regulator overflows only in intense storms.

2.5 According to the original estimate of the Metro Toronto authority, the WPCP has peak capacities of 11.8 $m^3/s$ (primary treatment and outfall) and 9.6 $m^3/s$ (secondary treatment). The authority has now revised the capacities of both primary and secondary treatment to 8.9 $m^3/s$. All the capacities are hypothetical values. It appears that an in-WPCP evaluation is

necessary if the true capacities are to be determined. Subject to the further investigation, this study assumes the capacities as originally estimated. The findings in the report are not affected by the revised capacities except that CSO Control Scheme 2 (paragraph 5.0) will not be feasible if the peak primary capacity should, in fact, be less than 11.8 $m^3/s$.

## 3.0 Analysis

Model simulation is used to estimate CSO statistics for existing and postulated conditions of catchments and sewers. All analyses use the precipitation data of the April-October 1979 season which is found to be representative of the average conditions in the precipitation history of the study area.

## 4.0 The Existing CSO Situation

4.1 With catchments and sewers in existing (1983) conditions, the regulators overflow in 26 storms, excluding the intense storm mentioned in paragraph 4.2. There are 64 rain events in the season. The estimated seasonal total volumes and pollutant loads, also excluding the intense storm, are as follows:

| | Overflow* | Detained by Tank |
|---|---|---|
| Volume ($m^3$) | 334,000 | 102,000 |
| Suspended solids (kg) | 63,000 | 16,000 |
| $BOD_5$ (kg) | 16,000 | 5,000 |
| Total phosphorus (kg) | 690 | 210 |
| Soluble phosphorus (kg) | 230 | 72 |
| Lead (kg) | 65 | 20 |
| Zinc (kg) | 112 | 35 |
| Copper (kg) | 41 | 13 |
| Cadmium (kg) | 2.6 | 0.8 |

(* includes overflow from existing tank)

The overflow volume is 20% and the detained volume 8% of the combined sewage yielded by the combined sewer area in the 26 storms.

Seasonal fecal coliform load is not given because it is a meaningless statistic. In a single storm, CSO fecal coliform load ranges from 80,000 billion to 670,000 billion organisms.

If CSO in the 26 storms is intercepted and treated at the Humber WPCP, the increase in WPCP seasonal treatment load will be 1/2%.

The impacts of CSO pollutant loads on the receiving waters will be studied by a separate TAWMS project.

4.2 The intense storm mentioned in paragraph 4.1 has a recurrence interval of 3.3 years, a total precipitation of 36.4 mm, and a maximum 1-hour precipitation of 28.1 mm. It produces a total overflow volume of 265,000 $m^3$, of which 160,000 $m^3$ overflow from the combined sewer area to the Black Creek and 105,000 $m^3$ overflow from the Berry Road regulator to the Humber river. The Berry Road overflow is attributed to wet-weather inflow/infiltration from the sanitary sewer area, not sewage from the combined sewer area. This is the only storm that causes the Berry Road regulator to overflow. The largest single-event overflow volume in all the other storms in the season is 41,000 $m^3$ only. It is concluded that the storm that causes the Berry Road regulator to overflow is intense and infrequent enough to be excluded from consideration in the development of CSO control schemes.

4.3 Under existing conditions, the WPCP has sufficient peak capacity to treat the wet-weather flow it receives. The combined sewer area requires a larger peak treatment capacity (0.0000298 $m^3$/person/s) than the sanitary sewer area (0.0000164 $m^3$/person/s). Their ratio is 1.8 to 1.0. Averaged over the season, however, the combined sewer area requires a smaller average unit treatment capacity than the sanitary sewer area. The respective figures are 0.526 $m^3$/person/d (including combined sewage) and 0.735 $m^3$/person/d. Their ratio is 1.0 to 1.4.

## 5.0 CSO Control Schemes

5.1 Five CSO control schemes are considered feasible. Each scheme is self-contained and is an alternative to the other four. Each scheme is analyzed for its effectiveness in CSO reduction against various capacities up to complete CSO elimination in the season (except in the intense storm). The maximum recurrence interval of storms in which the schemes may achieve complete CSO elimination is 1.8 years. The schemes are listed below in descending order of their relative merits.

| | | Order of Costs for Complete CSO Elimination<br>$ million |
|---|---|---|
| 1. | Detention of overflow at regulators. | 2.8 |
| 2. | Increase in combined sewage flow to WPCP by resetting regulators. | 3.5 |
| 3. | Schemes using runoff control: | |
| | (i) Detention tanks in local combined sewers. | 6.1 |
| | (ii) Disconnection of existing roof leaders. | 3.1 |
| | (iii) Separation of combined sewers. | 14.0 |

5.2 General notes on the schemes:

- The components of each scheme are defined in paragraph 5.3.

- Costs shown exclude costs of land, engineering services and ancillary works such as access road, instrumentation and landscaping of storage tank sites. Detailed engineering feasibility and costing will be studied by a separate TAWMS project.

- Flow detained in storage will be returned to the WPCP after a storm.

- Land for new storage tanks at the regulators is now zoned as green space. Change in use in some land is expected but land required for new storage is expected to be available.

- Stormwater runoff in each of Schemes 3(i), (ii) and (iii) is reduced by an equivalent of 20% of the combined sewer area. Schemes assuming more reduction (28%) of the combined sewer area are studied but the stormwater runoff reduced for each hectare of area reduced is not better than the results of Schemes 3(i), (ii) and (iii).

5.3 Comments on individual scheme:

- All capacity requirements given below refer to complete CSO elimination as defined.

Scheme 1: Detention of overflow at regulators.

- Scheme consisting of 2 new tanks. One tank (16,000 $m^3$) to intercept overflow from existing tank; one tank (35,000 $m^3$) to intercept all regulators overflowing to Black Creek.

- Cheapest, most practical, most reliable scheme.

Scheme 2: Resetting regulators.

- Scheme consisting of duplication of existing Metro Black Creek sanitary trunk sewer by a new sewer of 1.2 m diameter and 2.1 km length and 2 new tanks (16,000 $m^3$ and 15,000 $m^3$) at regulators.

- Regulators to be reset to increase wet-weather flow to WPCP from 2.43 to 4.43 $m^3/s$.

- Peak primary treatment capacity of WPCP assumed to be 11.8 $m^3/s$. Scheme not feasible if peak primary treatment capacity to be less than 11.8 $m^3/s$.

- The increased wet-weather flow to WPCP to receive primary treatment only.

- WPCP capacity stressed to the limit in wet weather.

- Possible disruption of golf course by construction of new sewer.

Scheme 3(i): Detention tanks in local combined sewers.

- Scheme consisting of 50 tanks of 300 $m^3$ each in local sewers and 29,000 $m^3$ of new storage at regulators.

- Systematic program for cleaning and maintenance of flow valves in local tanks required.

- No apparent serious disadvantage except high cost.

Scheme 3(ii): Disconnecting roof leaders.

- Scheme consisting of disconnection of roof leaders of 5,800 houses and provision of 29,000 $m^3$ of new storage at regulators.

- Scheme considered marginally feasible. Effectiveness of roof leader disconnection in flow reduction not well proven.

- No provision in existing sewer by-law for disposition of roof leaders, except in special cases.

- Public acceptance of large scale disconnection uncertain.
- Possible seepage of surface water into basements if disconnection done improperly.

Scheme 3(iii): Separation of combined sewers.

- Scheme consisting of new sewers in 248 ha of catchment for separation of road drainage from combined sewers and 29,000 $m^3$ of new storage at regulators.
- Extensive disruption of neighbourhood by sewer construction.
- Many years required for completing a sewer separation programs. Meantime, partially completed sewers not able to function fully.
- Runoff pollutants of the separated area still discharged to receiving waters. Seasonal SS load so discharged estimated at 67,000 kg, slightly more than the 63,000 kg of CSO SS load before sewer separation.
- Stormwater discharges from the new storm sewers not contaminated by sanitary sewage.

## 6.0 Basement Flooding Mitigation

A cursory analysis is carried out to explore the possibility of integrating some measures for basement flooding mitigation into CSO control schemes, although the analysis is outside the study scope. The results indicate that design conditions for flooding mitigation and CSO control are fundamentally different and the measures for the two objectives have to be provided independently of each other. The provisions for the two objectives do not augment each other.

## RECOMMENDATIONS

1. TAWMS should ascertain whether or not CSO control should be included in the watershed management plan.

2. If CSO control is required solely for bacterial control and not for other CSO parameters or pollutants, then TAWMS should consider whether high-rate disinfection can be a viable alternative to the CSO control schemes presented in this report.

3. The five alternative CSO control schemes should be considered in the following descending order of preference, subject to detailed feasibility study and costing:

   - Detention of overflow at regulators
   - Resetting regulators
   - Detention tanks in local sewers
   - Disconnecting roof leaders
   - Separation of combined sewers.

4. CSO control in less frequent storms (recurrence interval longer than 2 years) is not recommended.

5. Scheme 3(ii) using roof leader disconnections should be considered for adoption only after its effectiveness and reliability are proven by a large-scale pilot project.

6. The implementation of a CSO control scheme should be phased, preferably as follows:

| Scheme | Phase 1 | Phase 2 |
|---|---|---|
| - Detention of flow at regulators | Tank to intercept regulators | Tank to intercept Hyde Avenue tank |
| - Resetting regulators | Tank to intercept regulators | 1. Black Creek sewer duplication<br>2. Tank to intercept Hyde Avenue tank |
| - Detention tanks in local sewers | Tanks at 1st priority locations | Tanks at 2nd priority locations |
| - Disconnecting roof leaders | Depending on institutional considerations | |
| - Separation of combined sewers | Downstream sections of sewers | Upstream sections of sewers |

7. The implementation of a CSO control scheme should be accompanied by a performance monitoring program. The program should be planned scientifically and supervised properly.

8. The incompatibility of using a CSO control scheme to augment basement flooding mitigation or vice versa should be recognized.

## 1.0 INTRODUCTION

### 1.1 Background

The Combined Sewer Overflow Study presented in this report is one of the projects of the Toronto Area Watershed Management Strategy Study (TAWMS). The goal of TAWMS is to develop a comprehensive water quality management strategy for the Humber and the Don river watersheds (Figure 1.1).

Initial emphasis of TAWMS is on the Humber river watershed. This report will mention work done on this watershed only.

The Humber river watershed has a drainage area of about 897 square kilometres (Acres, 1984). Sub-watersheds in the upper reaches of the river are primarily rural or agricultural areas (Figure 1.1). Sub-watersheds in the lower reaches are highly urbanized and include Metropolitan Toronto.

Water qualities of the river were studied in a TAWMS project (Ministry of the Environment, 1983). Its conclusions indicated that:

- bacterial densities in the urbanized reaches were high and body-contact recreation often had to be restricted;
- there was continual enrichment of stream waters;
- some heavy metals, including cadmium, copper, lead and zinc, were often found at concentrations in excess of provincial water quality objectives; and
- the Black Creek, which is a tributary to the Humber river (Figure 1.1), showed the most degraded water quality.

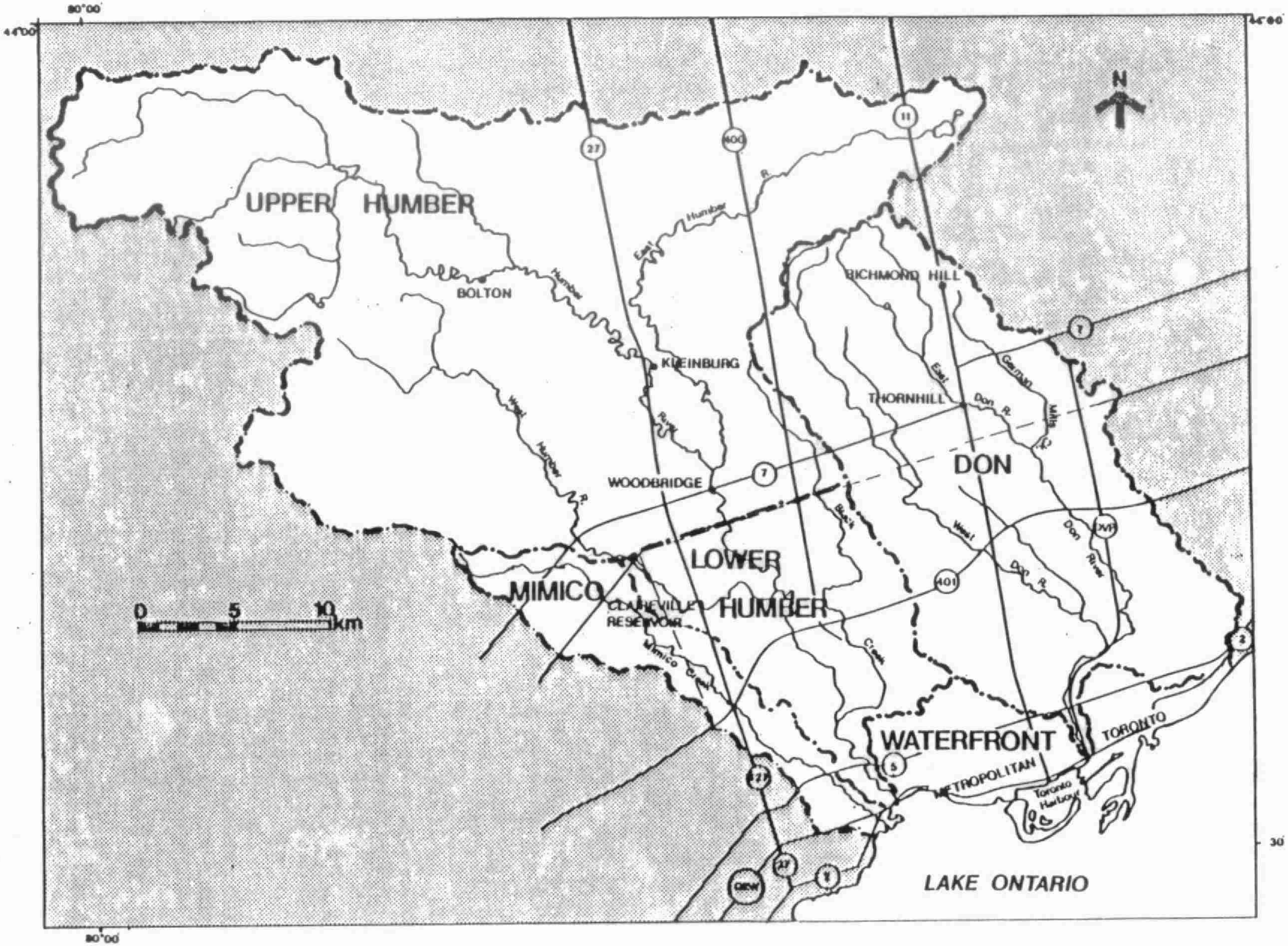



FIGURE 1.1: TORONTO AREA WATERSHED MANAGEMENT STRATEGY STUDY

In the light of the findings of the above preliminary study, TAWMS takes the following 3-part approach in pursuit of its goal:

- To investigate pollution sources and explore and evaluate means of controlling them;

- To examine impacts of pollution sources on the water qualities of the Humber river and predict water quality improvements due to pollution controls; and

- To develop a strategy for management of the water qualities of the river. The strategy will be developed using results of the investigations and having regards for economic, social, environmental and institutional effects of the strategy.

The Combined Sewer Overflow Study (CSO study) belongs to the first part of this approach. It started in 1983 after some preliminary field monitoring work was carried out in late 1982.

## 1.2 Organization of Report

The text of this report is written for a general readership. Technical matters are presented as simple as possible in the text. More complex technical discussions and details are placed in appendices.

The text, however, includes some introductory discussions that provide an underlying understanding of the nature, analysis and control of combined sewer overflow. The text also includes some literature review to illustrate how combined sewer overflow studies were carried out elsewhere in recent years. The purpose of the literature review is to share experiences of others and to provide a measure of the technical adequacy of the present CSO study. Readers may pass over those topics, if they so wish, without loss of continuity of the text.

## 1.3 Introductory Discussion of Combined Sewer Overflow

Combined sewers were laid in many urban areas developed before the 1960's to collect and transport both sanitary wastewater and stormwater surface runoff. Newer urban areas are served by two separate sewer systems, one for sanitary wastewater and one for stormwater runoff.

In a combined sewer, the dry-weather flow is basically sanitary wastewater but in wet weather, the flow is a mixture of both sanitary wastewater and stormwater runoff. This mixture is combined sewage. Its occurrence is intermittent and coincides with the occurrence of stormwater runoff. In the Toronto area, wet weather in the summer occurs at an average interval of about 3 days.

The flow rate of combined sewage varies from moment to moment in a storm and so does the proportion of sanitary wastewater and runoff in the flow. Figure 1.2 is an observed example demonstrating the variations in the flow rate and flow proportion. It can be seen that the combined sewage flow rate can be many times the dry-weather flow rate.

In a combined sewer network, provision often exists for some sewage to escape or overflow from the sewers via regulators under certain high flow conditions.

Regulated overflow is a means of ensuring that the water pollution control plant that intercepts the sewers will not be overloaded in wet weather. It is also a means of releasing excessive flow in an "orderly" manner to avoid sewage backing up in sewers with the result that sewage overflows indiscriminately from manholes and into basements of buildings. It is common practice to discharge the overflow to a watercourse.

An overflow regulator is a specially designed opening usually made at the side of a sewer. Its shape and level of setting determine the sewer flow rate (the threshold capacity) at which overflow will

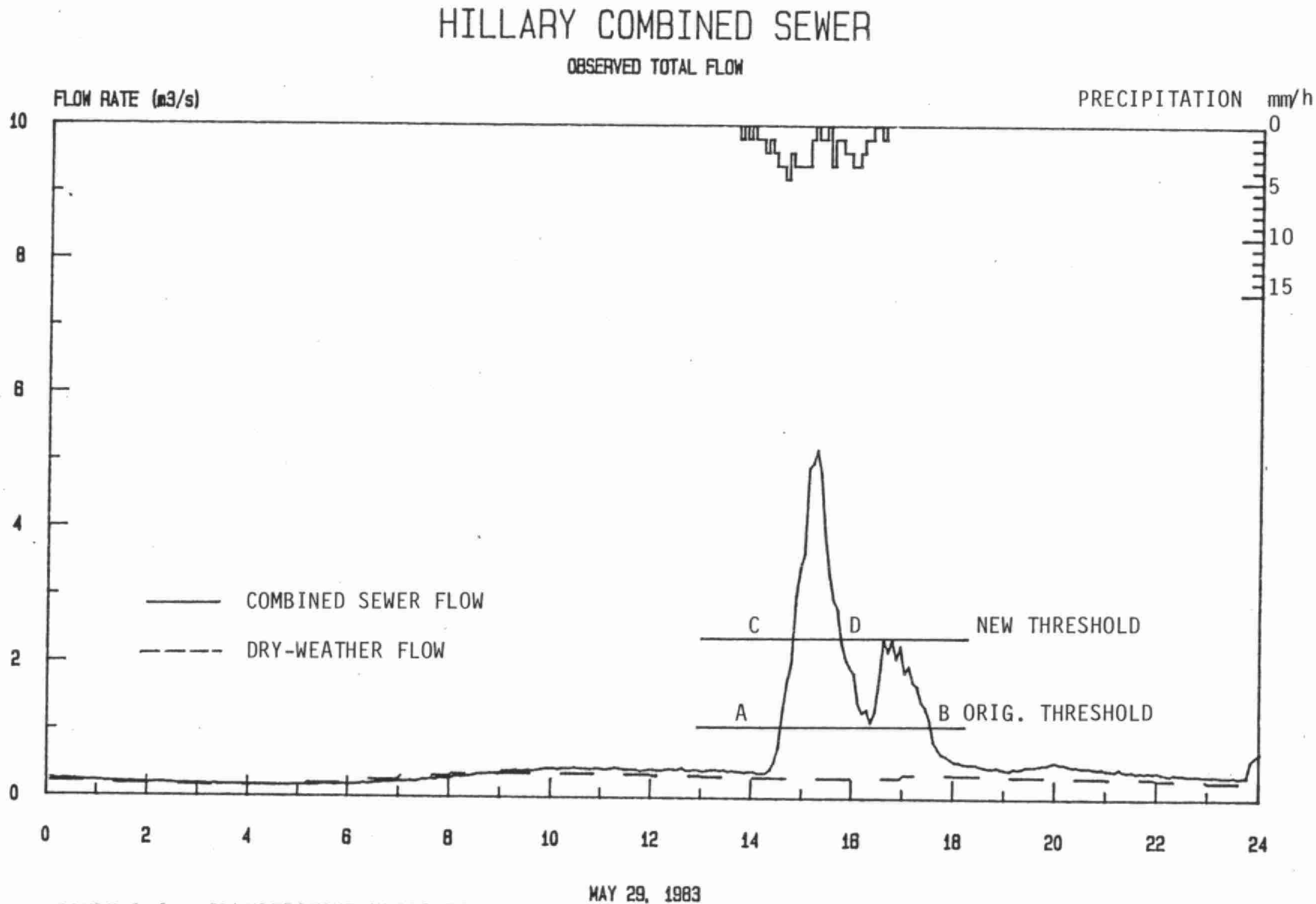



FIGURE 1.2 : ILLUSTRATIVE HYDROGRAPH

occur. Again using Figure 1.2 for illustration, if the threshold capacity is 1.0 $m^3/s$, overflow will occur between 14:30 hr. and 17:30 hr. and the volume of overflow is the area bounded by the curve and the line AB drawn at the threshold capacity. If the threshold capacity is increased to, say, 2.5 $m^3/s$, the overflow duration will be shortened to between 14:40 hr and 15:40 hr. The overflow volume will be the area bounded by the curve and the line CD. It is smaller than the previous volume.

A regulator is usually designed with an overflow capacity such that the sum of the overflow capacity and the threshold capacity is at least equal to the capacity of the sewer leading to the regulator. This design approach is to ensure that the regulator will not become a flow constriction to cause surcharge of the sewer upstream of the regulator.

The study of combined sewer overflow is closely related to the study of stormwater runoff and the sewer network. One major difference between a stormwater study and a combined sewer overflow study is that combined sewer overflow contains sanitary wastewater. Therefore, for some pollutants such as pathogenic bacteria, combined sewer overflow is a more potent polluter than runoff alone. Another major difference is that, while a storm produces runoff, the storm does not necessarily produce sufficient combined sewage to cause overflow.

## 2.0 OUTLINE OF STUDY

### 2.1 Study Objective and Scope

The CSO study was initiated with the objective of providing TAWMS with CSO information of the Humber sewershed for the development of TAWMS's water quality management strategy for the watershed.

The CSO study area is shown in Figure 2.1. The required scope of the study comprises the following:

1. To collect data required for the CSO study; snow data not required;

2. To characterize the catchments and sewers of the CSO system (the study system);

3. To estimate CSO pollution from the existing study system; and

4. To develop possible schemes for control of the CSO pollution and to evaluate the effectiveness of the schemes in CSO reduction.

Moreover, the study requires that computer model simulation will be employed in the CSO analysis. The analysis work will be carried out with sufficient detail to produce CSO statistics to meet the study objective, but hydraulic analysis of local trunk sewers will not be required. Pollutants to be studied will be suspended solids, total and soluble phosphorus, lead, zinc, copper, cadmium, fecal coliforms and $BOD_5$. The CSO analysis period will be a selected season of a selected year. The season will be selected in which CSO is typically most predominant in the year. The year will be selected in which precipitation will be representative of historical average conditions. All CSO control schemes will be subject to two constraints, namely, there will be no increase in sewage level (hence no aggravation of basement flooding) in the sewer network upstream of overflow regulators; and the peak capacities of the

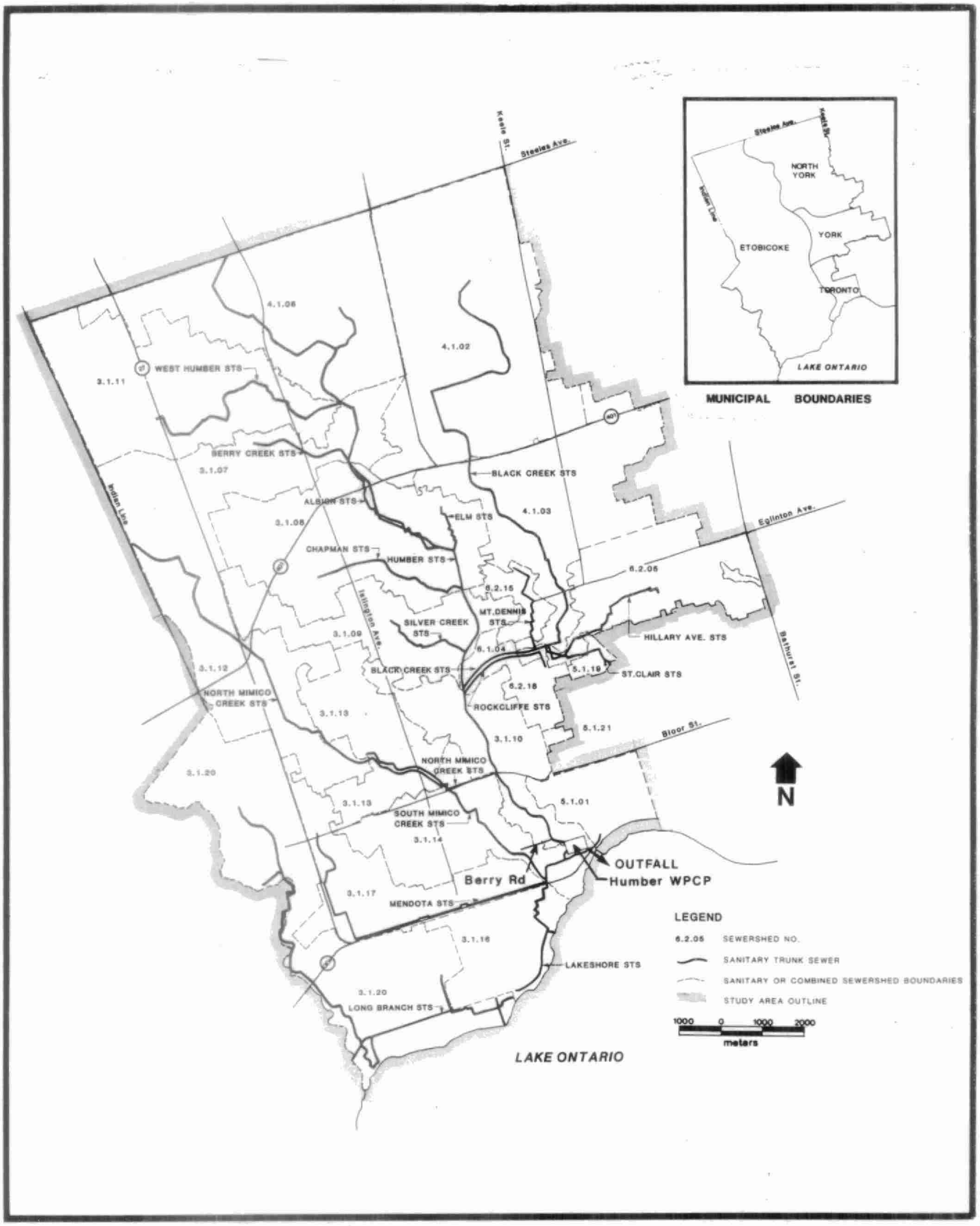



FIGURE 2.1 : HUMBER SEWERSHED : SANITARY AND COMBINED SEWER KEY MAP

existing Humber Water Pollution Control Plant (WPCP) will not be exceeded. CSO control schemes will be developed to the conceptual layout stage so that engineering feasibility and costs of the control schemes may be studied by a separate TAWMS project.

## 2.2 Overview of the Study

The action plan developed for the CSO study is outlined in Figure 2.2 and briefly described below.

| Activity | Purpose |
|---|---|
| Collect catchment and sewer data | To understand the study system. To configure the system for hydrologic, hydraulic and pollutant loading computations. |
| Collect flow data | To observe response of the study system to wet and dry weather. To derive parameter values for estimating flow and pollutant loadings. To provide observed data to validate simulation results. |
| Collect precipitation data | To provide a link between observed precipitation and observed flow. |
| Model simulation | To obtain methodically statistics and trends of CSO loading changes in response to control. |
| Provide results for water quality study | To provide input and link for evaluating CSO impacts on receiving waters. |
| Draw up conceptual control schemes | To indicate conceptual layout of CSO control schemes for examining feasibility and costs and for future reference for design and operation of schemes. |

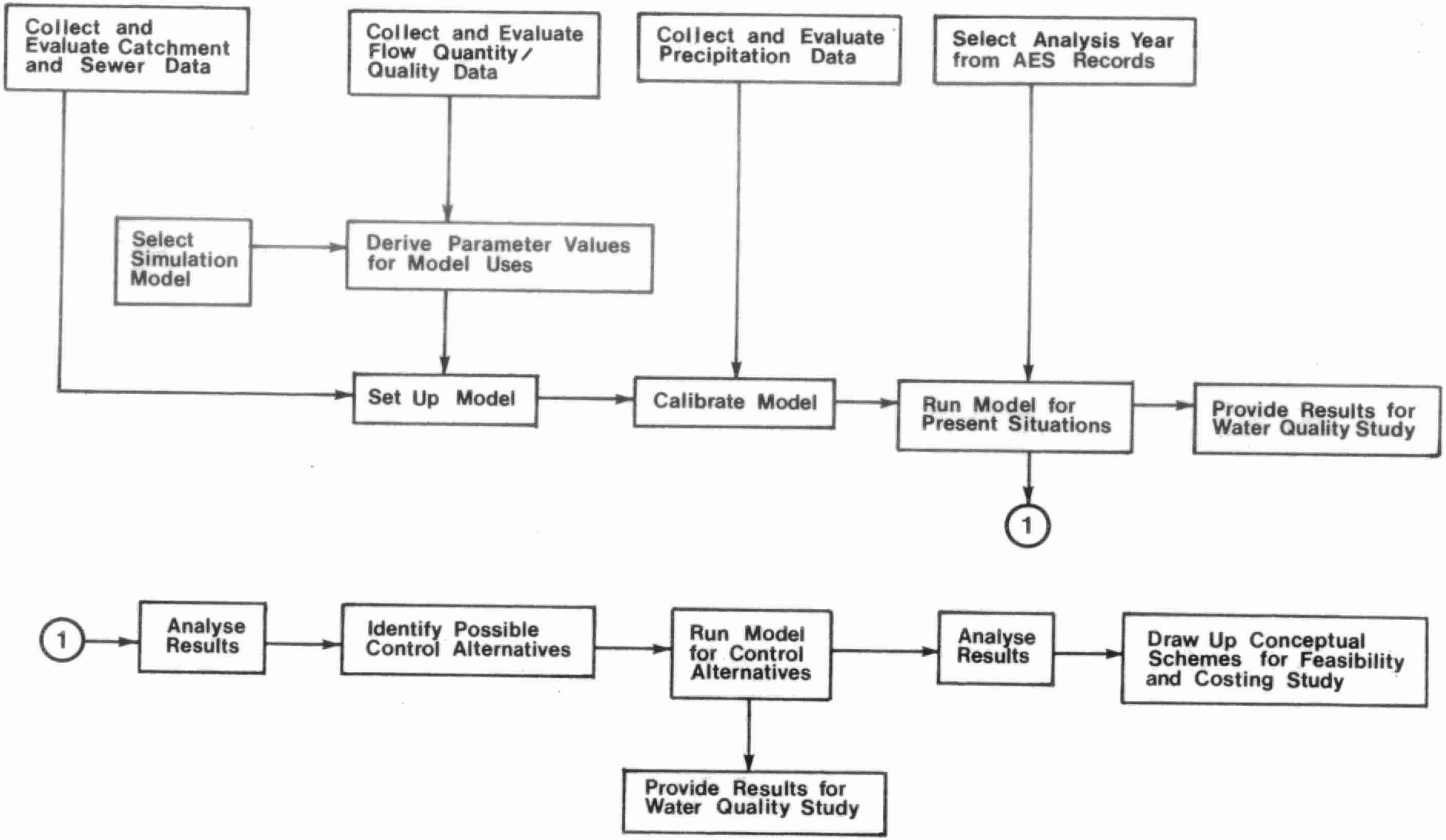



FIGURE 2.2 : STUDY ACTION PLAN

# 3.0 THE CATCHMENTS AND SEWERS

## 3.1 Description of Study Area

The study area (Figure 2.1) includes partly or wholly 4 cities of Metro Toronto, namely Etobicoke, North York, Toronto and York. Three areas in the study area are served mainly by combined sewers. They are marked 6.2.05 (Hillary Ave), 6.2.15 (Mt. Dennis) and 6.2.18 (Rockcliffe). Their total area is 1,331 ha of which 1,246 ha are combined sewer area and the rest is separate sewer area. The areas are connected to the Humber Sanitary Trunk Sewer (STS) which flows to the Humber WPCP. The remainder of the study area is served by separate sanitary and stormwater sewers. The sanitary sewer area has an area of 17,418 ha and is also connected to the Humber STS and the Humber WPCP. It was included in the study so that the total hydraulic loading on the Humber STS and the WPCP can be determined in the case of CSO control measures affecting these facilities.

The study area has a gentle terrain sloping southward in the general direction of the Humber river. The area in the lower reach of the Black Creek, where the combined sewer area exists, dips in a southwesterly direction as the Black Creek turns its way to join the Humber river.

About half of the combined sewer area is pre-World War II development. The other half was developed or redeveloped in the 1960's or early 1970's. Generally, building lots have short frontage. Stormwater runoff from roads, sidewalks and driveways is collected by gutters and underground sewers. Grassed swales or curb-side turf plots are uncommon. About two-thirds of the houses have their roof leaders connected to sewers. The other roof leaders discharge to ground surfaces. Streets are mostly smooth and well maintained. The top layer of soil in unpaved areas is mainly silty or clayey.

The sanitary sewer area is mainly an established sub-urban development. Its topography and surface characteristics were not of direct interest to this study because its stormwater runoff is not collected by the study sewer system. Wet-weather inflow/infiltration

(I/I) to the sanitary sewers, however, was accounted for as explained in Section 4.6.

## 3.2 Catchment Data

Most catchment data used were abstracted from a TAWMS data report (Gartner Lee Associates, 1983). The study area was divided into 3 main combined sewer catchments and 17 sanitary sewer catchments in anticipation of data needs for model simulation. The data are summarized in Table 3.1, several points of which are worth noting. The combined sewer area is only 7% of the whole study area. Its population density is more than double the average of the study area. Its wastewater production rate per person is only about half of the average of the sanitary sewer area's. It has a smaller percentage of industrial land use (Table 3.2) but a much larger component of low/medium residential use and commercial use.

In summary, the development characteristics of the combined sewer area are notably different from those of the sanitary sewer area, possibly because the combined sewer area is generally a few decades older (Gardner Lee Associates, 1983).

For model simulation purposes, the combined sewer catchments were further divided into subcatchments and data were compiled for the size of catchment or subcatchment, length of overland flow path, ground slope, length of street curb, percentage of impervious surface, population density and wastewater production rate for each land use. Similarly, data were compiled for each sanitary sewer catchment and the data included catchment size, population density and wastewater production rate for each land use.

An anomaly in the wastewater production rates was found in the original data (Gartner Lee Associates, 1983), but it had been corrected in the data presented above. The anomaly was that the sum of wastewater quantities calculated from the data was much larger than the average dry-weather flow (DWF) recorded at the Humber WPCP. In principle, the DWF should be the larger because it is the total of wastewater produced and groundwater infiltrated into the sewers. To correct the anomaly, additional water consumption data

TABLE 3.1

SUMMARY OF CATCHMENT DATA

| | Area (ha) | Area (% of Study Area) | Population | Population Density (No./ha) | Wastewater Production Rate (2) m^3/d | m^3/p/d | m^3/ha/d |
|---|---|---|---|---|---|---|---|
| (A) Combined Sewer Area (1) | | | | | | | |
| Hillary | 951.1 | 5.1 | 61,716 | 65 | 16,027 | 0.260 | 16.9 |
| Mt. Dennis | 183.5 | 1.0 | 9,564 | 52 | 3,237 | 0.339 | 17.6 |
| Rockcliffe | 196.1 | 1.1 | 9,994 | 51 | 2,553 | 0.256 | 13.0 |
| Subtotal of (A) | 1,330.7 (1) | 7.1 | 81,274 | 61 | 21,817 | 0.268 | 16.4 |
| (B) Black Creek San. Area | 4,027.9 | 21.5 | 158,981 | 39 | 64,003 | 0.403 | 15.9 |
| (C) Remaining San. Area | 13,390.2 | 71.4 | 289,950 | 22 | 185,116 | 0.638 | 13.8 |
| (D) All Sanitary Area (i.e. B + C) | 17,418.1 | 92.9 | 448,931 | 26 | 249,119 | 0.555 | 14.3 |
| (E) Study Area (i.e. A + D) | 18,748.8 | 100.0 | 530,205 | 28 | 270,936 (3) | 0.511 | 14.5 |

Notes:

(1) Including sanitary area of 84.7 ha.
(2) Including residential, commercial and industrial uses.
(3) Not including 17,280 m^3/d sewage from Mississauga.
Total including Mississauga was 288,216 m^3/d.

TABLE 3.2

LAND USE DISTRIBUTION (1)

| | Residential Low/medium Density | Residential High Density | Commercial | Industrial | "Others" (2) |
|---|---|---|---|---|---|
| (A) Combined Sewer Area (Hillary + Mt. D. + Rock.) | 71% | 2% | 7% | 11% | 9% |
| (B) Black Creek San. Area | 43% | 6% | 3% | 20% | 28% |
| (C) Remaining San. Area | 39% | 3% | 4% | 26% | 28% |
| (D) All Sanitary Area (i.e. B + C) | 40% | 4% | 4% | 24% | 28% |
| (E) Study Area (ie. A + D) | 42% | 4% | 4% | 24% | 26% |

Note:

(1) Expressed in per cent as land use area divided by catchment area.
(2) Includes institutional uses and open spaces.

were obtained from a municipal water accounts department and the wastewater quantities were recalculated as explained in Appendix A1. The revised data compared well with observed DWF as explained in Section 4.2 and were adopted.

### 3.3 Description of the Combined Sewer System

A layout of the trunk combined sewers and the locations of the major overflow regulators is shown in Figure 3.1. Combined sewage flows to the Humber WPCP via the Black Creek STS, the Rockcliffe STS and the Humber STS. In wet weather, combined sewage up to 2.43 $m^3/s$ (5.8 times of the DWF of the combined sewer area) is allowed to flow to the WPCP. Flow in excess of this limit is partly diverted to an existing detention tank on Hyde Avenue and the remainder overflows to the Black Creek at the Site 3, Mt. Dennis and Rockcliffe overflow regulators. The Hyde Avenue tank has a capacity of 7,800 $m^3$. When full, the tank also overflows to the Black Creek. The detained sewage is returned to the WPCP via the Black Creek STS after a storm.

The tank is an underground concrete tank with an annexed operating room on the ground. Normally, the valve of the underdrain for return flow is open and the tank is empty at the onset of a storm. When inflow first arrives at the tank, the rising water level closes the valve automatically. After a storm, the valve is opened manually to drain the tank. The draining of a full tank takes about 11 hours.

Only the Hillary catchment diverts excessive flow to the tank. The catchment's flow to the WPCP in both dry and wet weather follows the dashed lines. In wet weather, excessive flow diverted to the tank follows the dotted lines. The diversion is controlled by regulators other than those mentioned earlier. Since the latter set of regulators does not divert any combined sewage to receiving waters directly, these regulators are of no direct interest to this report and will not be mentioned further.

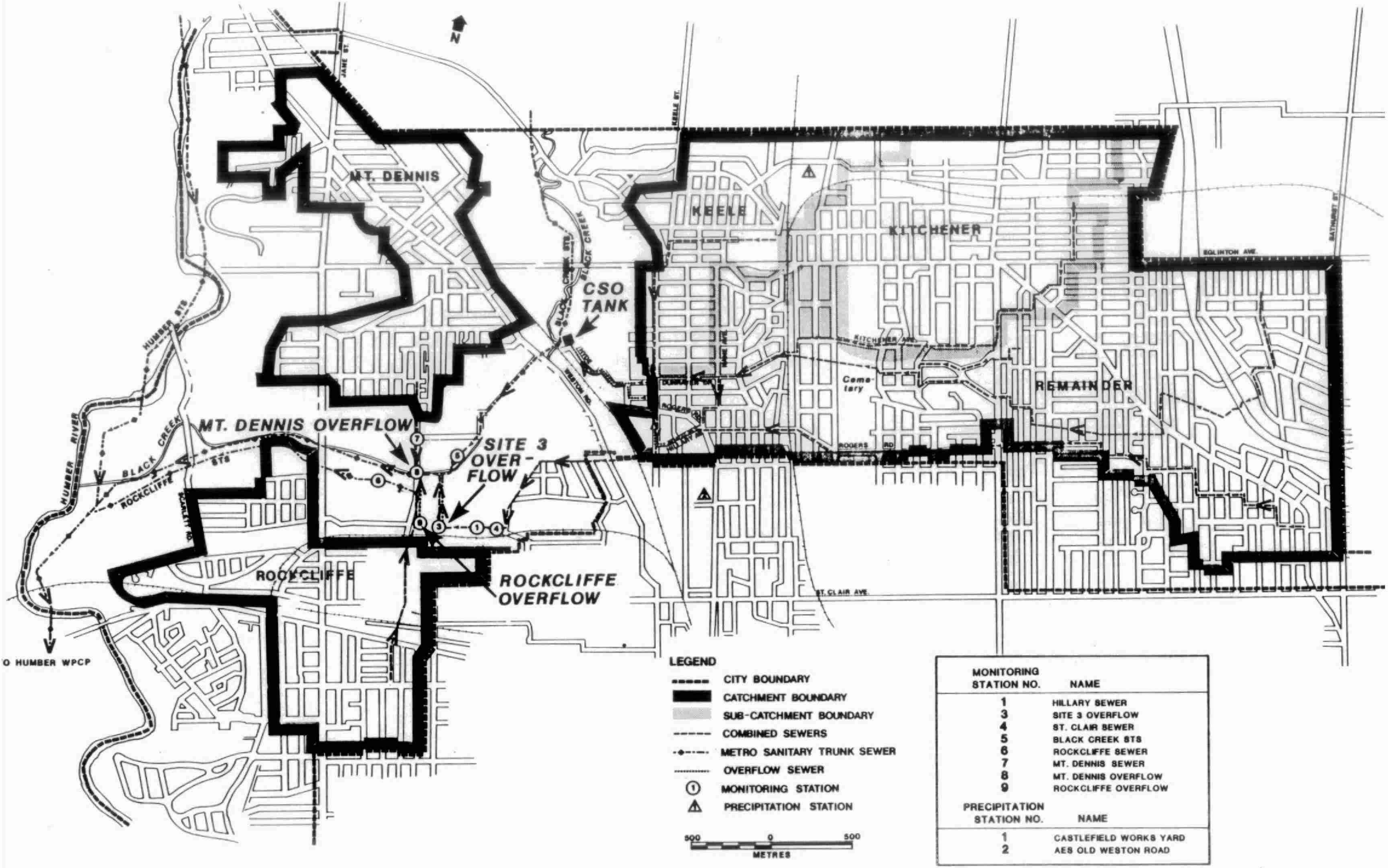



FIGURE 3.1 : MAP OF COMBINED SEWER AREA

Overflow to the Black Creek at regulators will occur when flow in the sewers at the regulator points reach the following rates (threshold capacities) determined from sewer engineering details:

| Regulator | Threshold Capacity ($m^3/s$) | DWF at Regulator ($m^3/s$) |
|---|---|---|
| Site 3 | 1.64 | 0.312 |
| Mt. Dennis | 0.32 | 0.060 |
| Rockcliffe | 0.47 | 0.050 |

### 3.4 The Berry Road Regulator and the Humber WPCP

The Berry Road regulator is installed in the Humber STS at Berry Road (Figure 2.1). It discharges to the Humber river. It is the last regulator to limit the flow to the Humber WPCP.

This regulator may overflow in one or both of two circumstances which are illustrated in Figure 3.2. Overflow will occur if the flow at the upstream end of the regulator exceeds the threshold capacity of $10.2 m^3/s$. Overflow will also occur if the operator of the Humber WPCP closes some of the gate valves of the WPCP to restrict the flow entering the WPCP. In this circumstance, the restricted flow will back up in the Humber STS and overflow at the regulator.

It was assumed in this study that the allowable peak flow to the WPCP was equal to the peak primary treatment capacity of 11.8 $m^3/s$ and that the primary effluent in excess of the secondary treatment capacity of 9.6 $m^3/s$ bypassed the secondary treatment process and was discharged to the effluent outfall, after chlorination. The peak capacity of the effluent outfall is 11.8 $m^3/s$. The data, illustrated in Figure 3.3, were supplied by the Metro Toronto authority. The design peak primary and peak secondary treatment capacities were defined as the maximum capacities above which effluent qualities begin to deteriorate. (Metro Toronto, 1984).

In October 1985, after the draft of this report was prepared, the Metro Toronto authority revised the peak primary treatment capacity from 11.8 $m^3/s$ to 7.8 $m^3/s$ (if primary treatment is not followed

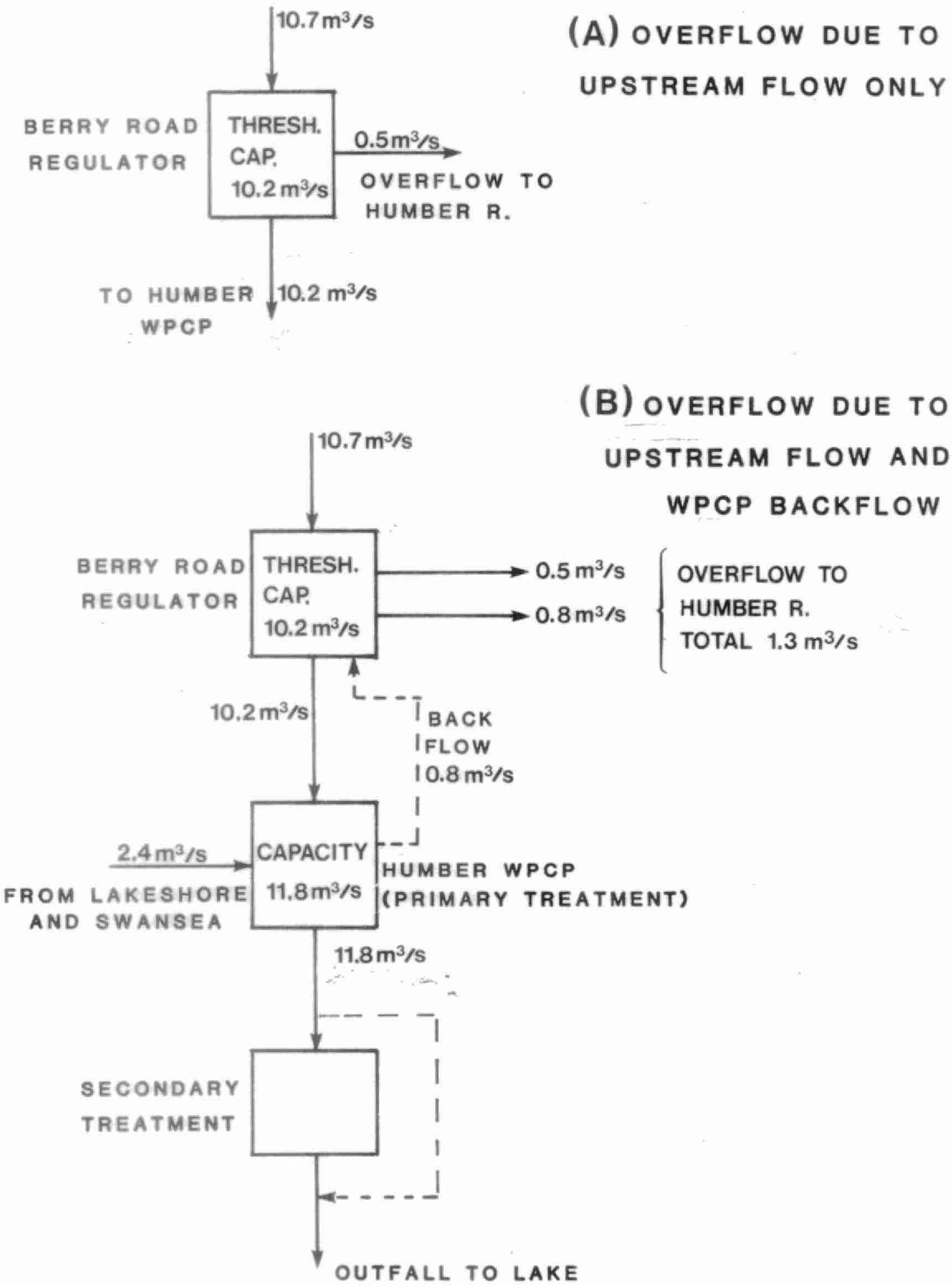


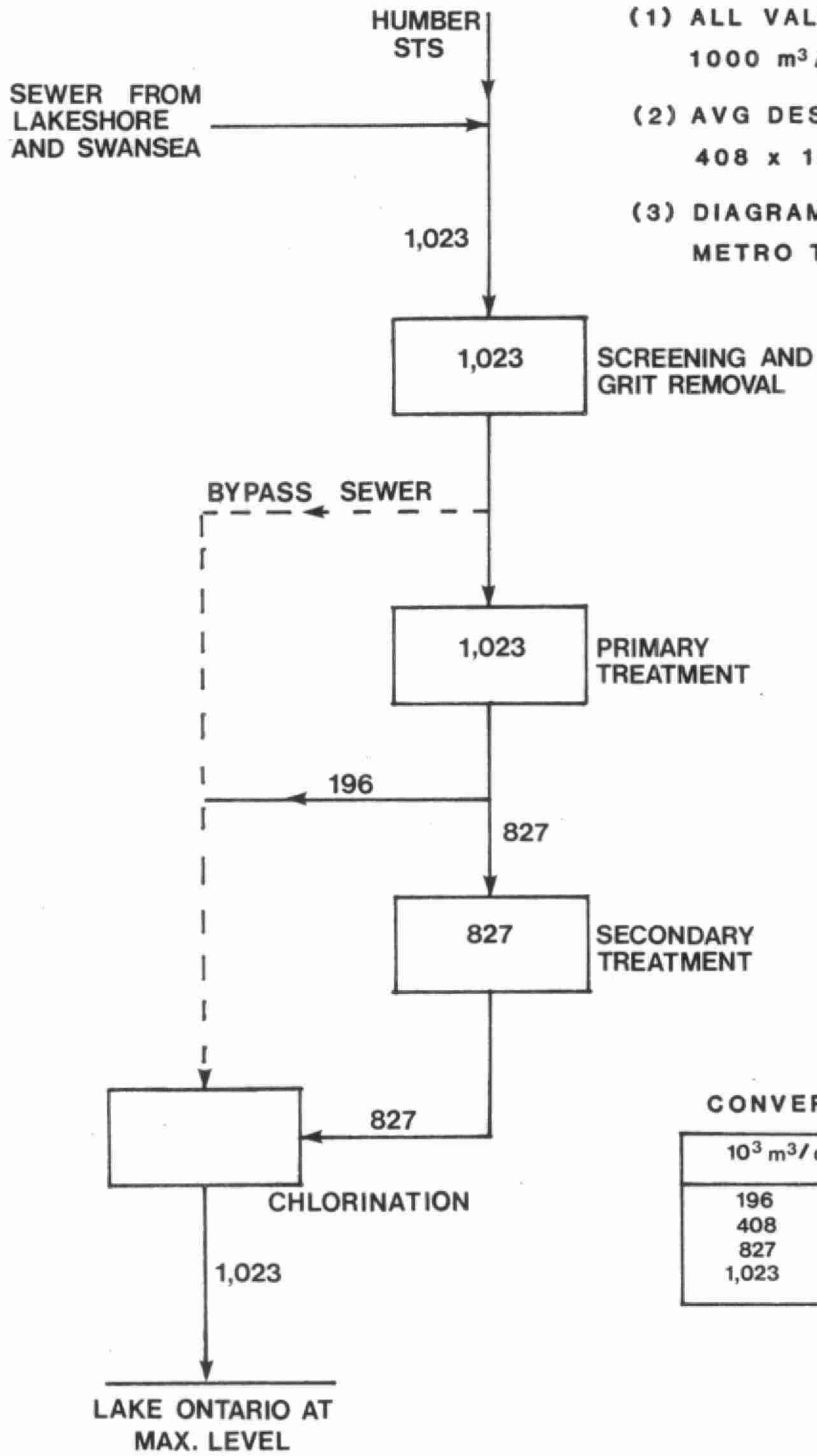

FIGURE 3.2: CAUSES OF OVERFLOW AT BERRY ROAD REGULATOR

NOTES:

(1) ALL VALUES ARE IN 1000 $m^3/d$.

(2) AVG DESIGN DWF IS 408 x $10^3$ $m^3/d$.

(3) DIAGRAM ABRIDGED FROM METRO TORONTO DRAWING.




CONVERSION TABLE

| $10^3$ $m^3/d$ | $m^3/s$ |
|---|---|
| 196 | 2.3 |
| 408 | 4.7 |
| 827 | 9.6 |
| 1,023 | 11.8 |

FIGURE 3.3: DESIGN PEAK SEWAGE FLOW OF HUMBER WPCP

by secondary treatment) and both the primary and secondary treatment capacities to 8.9 $m^3/s$ (if primary treatment is followed by secondary treatment). The Metro Toronto authority indicated that the revision was aimed at maintaining the effluent quality at all time. The revision would render CSO Control Scheme 2 (Section 7.3) infeasible, but no other conclusion in the report would be nullified.

An anomaly in the revision is observed in that the Humber WPCP would have a smaller throughput capacity if the WPCP was operated as a primary treatment plant than if it was operated as a secondary treatment plant. Since both the original and the revised capacities are hypothetical values and they conflict with each other, an in-WPCP evaluation is recommended if CSO Control Scheme 2 is to be considered for adoption. In the meantime, the original capacities were assumed in the study.

### 3.5 Sewer Data

Numerical data needed for computational analysis were mostly abstracted from the TAWMS data report (Gartner Lee Associates, 1983). The data included sewer sizes and invert elevations.

### 3.6 Minor Combined Sewer Catchments

Two minor combined sewer areas exist in catchments No. 5.1.19 (Keele Street/St. Clair Avenue) and 5.1.21 (Ardagh Street/Jane Street) shown in Figure 2.1. These two areas are outside the Hillary, Mt. Dennis and Rockcliffe catchments. As the two areas together are only 13.4 ha, they were regarded as minor local cases and were ignored in the study.

## 4.0 FLOW AND PRECIPITATION DATA

### 4.1 Data Collection Program

Dry-weather flow (DWF) and wet-weather flow (WWF) were monitored and sampled at selected locations of the sewer network for purposes shown in Table 4.1. The locations of the field stations are indicated in Figure 3.1.

Data collection efforts for the combined sewer area were concentrated on the Hillary catchment which makes up 72% of the combined sewer area. Four stations, namely, the inlet and the overflow weir of the Hyde Avenue tank and Sites 1 and 3 of the Hillary sewer, were needed to completely define the combined sewage flow of the catchment. The DWF at Site 1 was the total DWF of the catchment.

The Mt. Dennis and Rockcliffe stations were monitored to provide supplementary DWF quality data and CSO information.

Station 5 at the outlet of the Black Creek sanitary sewer area was used to provide data representative of DWF qualities of the entire sanitary sewer area of the study. Additionally, the data from this station, together with daily flow data obtained from the Humber WPCP log sheets, were used to determine the DWF quantities as well as dry and wet-weather infiltration rates of the sanitary sewer area.

The St. Clair Avenue sanitary sewer serves a catchment of only 47 ha. It was monitored and sampled because it receives flow from a number of meat packaging and protein recovery plants.

Precipitation was gauged at the Castlefield Works Yard station.

TABLE 4.1

FIELD DATA COLLECTION PROGRAM

| Location of Station | Station I.D. No. | DWF Quan. | DWF Qual. | WWF Quan. | WWF Qual. | Use of Data |
|---|---|---|---|---|---|---|
| | | Type of Data Collected | | | | |
| Castlefield Works Yard | | | | | | Precipitation data to correlate with observed combined flow. |
| Hyde Ave Tank Inlet | | | | X | | Part of Hillary WWF; tank utilization; model calibration. |
| Hyde Ave. Tank Overflow | 2 | | | X | | As above; overflow information. |
| Site 3 Regulator | 3 | | | X | | Part of Hillary WWF;overflow information. |
| Hillary Sewer (After Site 3) | 1 | X | X | X | X | Part of Hillary WWF; total DWF of Hillary. |
| Mt. Dennis Sewer | 7 | X | | | | Supplementary DWF quality information of combined sewer area. |
| Rockcliffe Sewer | 6 | X | | | | As above. |
| Mt. Dennis Regulator | 8 | | | X | | Check of overflow events. |
| Rockcliffe Regulator | 9 | | | X | | As above. |
| Black Creek San. Sewer Area | 5 | X | X | | | Representative DWF information of sanitary area. |
| St. Clair Ave. Sewer | 4 | X | X | | | Special industrial wastewater. |

Note: DWF = Dry-weather flow
WWF = Wet-weather flow

Most field data were collected in the period between April and October, 1983, although some DWF data were obtained in the fall of 1982. A summary of the instrumentation and data collection protocol is in Appendix B1.

All samples were analyzed by the Ministry of the Environment laboratory in Rexdale.

## 4.2 DWF Quantities

In calculating DWF, a dry-weather day was defined as a day without precipitation in the preceding 24 hours.

DWF data of 27 days were collected from Site 1 and analyzed to obtain the daily average flow, the ratios of flow variations in the days of the week and the ratios of flow variations in the hours of the day. These ratios were useful for synthesizing the DWF baseline of a combined sewage hydrograph whose DWF baseline could not be measured directly on a wet day. The observed average DWF at Site 1 was 26,946 $m^3/d$ or 0.437 $m^3$/person/d. Detailed data are in Appendix B2.

Similarly, 14 days of observed DWF data were collected at Site 5. The average DWF was 86,618 $m^3/d$ or 0.545 $m^3$/person/d. Detailed data are in Appendix B2.

The Humber WPCP data are summarized in Table 4.2. The data marked with an asterisk were considered as outliers because their values were incongruously smaller than the average of the remaining data set. The average DWF at the WPCP was 373,800 $m^3/d$ which included the DWF of 17,280 $m^3/d$ from the City of Mississauga discharged to the North Mimico Creek STS. Excluding the Mississauga flow, the average DWF at the WPCP was 356,520 $m^3/d$ or 0.672 $m^3$/person/d. It is clear that CSO control may consider utilizing spare primary treatment capacity but not spare secondary treatment capacity.

TABLE 4.2

RECORDED HUMBER WPCP DRY WEATHER FLOW

| | 1980 | | 1981 | | 1982 | | 1983 | | 4-Year |
|---|---|---|---|---|---|---|---|---|---|
| | No. of Dry Days | Avg Daily DWF | No. of Dry Days | Avg Daily DWF | No. of Dry Days | Avg Daily DWF | No. of Dry Days | Avg Daily DWF | Avg Daily DWF# |
| January | 22 | 348.1 | 28 | 261.9* | 18 | 336.8 | | | 342.4 |
| February | 24 | 326.5 | 10 | 259.8* | 18 | 334.4 | | | 330.5 |
| March | 16 | 320.7 | 24 | 246.1* | | | | | 320.7 |
| April | 8 | 350.9 | 17 | 252.3* | 18 | 392.4 | | | 371.7 |
| May | 20 | 333.6 | 13 | 216.6* | 16 | 364.6 | 6 | 371.8 | 356.7 |
| June | 11 | 408.8 | 14 | 239.9* | 12 | 415.3 | 18 | 362.3 | 395.5 |
| July | 12 | 359.0 | 16 | 401.2 | 24 | 338.7 | 23 | 325.9 | 356.2 |
| August | 20 | 391.1 | 13 | 411.6 | 13 | 309.7 | 14 | 372.2 | 371.1 |
| September | 17 | 427.1 | 10 | 380.3 | | | 18 | 352.6 | 386.7 |
| October | 11 | 456.4 | 10 | 356.2 | | | 14 | 342.2 | 384.9 |
| November | 16 | 367.8 | 23 | 383.0 | | | | | 375.4 |
| December | 19 | 363.5 | 16 | 331.5 | | | | | 347.5 |

For Season of April - October In 1980 - 1983#

Average Daily DWF 373.8 x 10^3m^3/d (4.33m^3/s)

Standard Deviation 36.1 x 10^3m^3/d

Notes:

(1) All volumes in 10^3m^3/d; unless stated otherwise.
(2) Data taken from WPCP operation records.
(3) * Data not used.
(4) # Excluding data marked with *.

There are two similarities between the observed DWF (Table 4.3) and the wastewater production rates (Table 3.1) mentioned in Section 3.2. One similarity is that the observed DWF of the combined sewer area (0.447 $m^3$/person/d) is about half of that of the sanitary sewer area (0.713 $m^3$/person/d) as the wastewater production rate of the combined sewer area (0.268 $m^3$/person/d) is half of that of the sanitary sewer area (0.555 $m^3$/person/d). The other similarity is that the observed DWF of the Black Creek sanitary sewer area and of the remaining sanitary sewer area are in the ratio of 1.0 to 1.5 (i.e. 0.545 $m^3$/person/d to 0.806 $m^3$/person/d) as the wastewater production rates of the same two areas are in the ratio of 1.0 to 1.6 (i.e. 0.403 $m^3$/person/d to 0.638 $m^3$/person/d). The similarities substantiate the reasonableness of the revised wastewater production data.

The dry-weather infiltration (Table 4.3) was 40.5% of the DWF in the combined sewers and ranged from 20.8% to 26.1% in the sanitary sewers. The higher percentage for the combined sewers was probably due to the combined sewers being older. However, 40.5% is not excessively high, because it is equivalent to 0.179 $m^3$/person/d only and is well within the range from 0.212 $m^3$/person/d to 0.593 $m^3$/person/d of dry-weather infiltration recommended for sewer design (Ministry of Environment, 1984).

### 4.3 DWF Qualities

DWF sample results are shown in Table 4.4. Details are in Appendix B2. The cadmium results shown in the Table were taken from the supplementary sampling in 1983 at the Mt. Dennis and Rockcliffe sewers, because samples collected in the Hillary catchment in 1982 were not analyzed for cadmium.

TABLE 4.3

COMPARISON OF DWF QUANTITIES

| Catchment | Area (ha) | Population | Wastewater Production (m^3/d) | Observed DryWeather Flow (DWF): Total DWF (m^3/d) | Observed DryWeather Flow (DWF): m^3/person/d | Observed DryWeather Flow (DWF): m^3/ha/d | Dry Weather Infiltration (2) |
|---|---|---|---|---|---|---|---|
| (A) Combined Sewer Area | | | | | | | |
| Hillary | 951.1 | 61,716 | 16,027 | 26,946 | 0.437 | 28.3 | 40.5% |
| Mt. Dennis | 183.5 | 9,564 | 3,237 | 5,184 (4) | 0.542 | 28.3 | 40.5% (3) |
| Rockcliffe | 196.1 | 9,994 | 2,553 | 4,320 (4) | 0.432 | 22.0 | 40.5% (3) |
| Sub-Total of (A) | 1,330.7 (1) | 81,274 | 21,817 | 36,288 | 0.447 | 27.3 | 40.5% |
| (B) Black Creek San. Area | 4,027.9 | 158,981 | 64,003 | 86,618 | 0.545 | 21.5 | 26.1% |
| (C) Remaining San. Area | 13,390.2 | 289,950 | 185,116 | 233,614 (6) | 0.806 | 17.4 | 20.8% |
| (D) All Sanitary Area (i.e. B + C) | 17,418.1 | 448,931 | 249,119 | 320,232 | 0.713 | 18.4 | 22.2% |
| (E) Study Area (i.e. A+D) | 18,748.8 | 530,205 | 270,936 | 356,520 (5) | 0.672 | 19.0 | 24.0% |

Notes:
(1) Including sanitary area of 84.7 ha.
(2) Expressed as (Obs. DWF-TWP)/Obs. DWF.
(3) Assumed same as Hillary.
(4) Derived value. Calculated as TWP/(1-.405).
(5) WPCP record of 373,800 m^3/d less 17,280 m^3/d from Mississauga.
(6) Derived value. Calculated as 356,520 - 36,288 - 86,618.

TABLE 4.4

DWF QUALITIES

| Hillary Catchment Station No. 1 | | BOD5 | RSP | PPUT | PPO4 | CUUT | PBUT | ZNUT | CDUT (1) |
|---|---|---|---|---|---|---|---|---|---|
| Avg. DWF(m^3/s) | 0.312 | | | | | | | | |
| Avg. Conc.(mg/l) | | 208.9 | 245.4 | 5.42 | 2.42 | 0.16 | 0.06 | 0.24 | .014 |
| No. of Samples | | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 12 |
| Std. Dev. (mg/l) | | 75.1 | 92.4 | 2.66 | 0.83 | 0.11 | 0.03 | 0.10 | .007 |
| **Black Creek San. Area Station No. 5** | | | | | | | | | |
| Avg. DWF(m^3/s) | 1.000 | | | | | | | | |
| Avg. Conc.(mg/l) | | 191.9 | 173.8 | 5.66 | 2.58 | 0.36 | 0.04 | 0.17 | |
| No. of Samples | | 14 | 14 | 14 | 14 | 14 | 14 | 13 | |
| Std. Dev. (mg/l) | | 52.4 | 44.3 | 2.94 | 0.89 | 0.13 | 0.02 | 0.05 | |
| **St. Clair Ave. Sewer Station No. 4** | | | | | | | | | |
| Avg. DWF(m^3/s) | 0.017 | | | | | | | | |
| Avg. Conc.(mg/l) | | 2077 | 1067 | 29.2 | 19.4 | 0.07 | 0.029 | 0.28 | |
| No. of Samples | | 9 | 9 | 8 | 9 | 9 | 9 | 9 | |
| Std. Dev. (mg/l) | | 547 | 639 | 6.95 | 5.67 | 0.032 | 0.014 | 0.037 | |

Note:

(1) Data from Mt. Dennis and Rockcliffe Catchments

RSP = Suspended Solids    ZNUT = Zinc.    PPO4 = Filtered Reactive Phosphorus.

CUUT = Copper.    PBUT = Lead.    PPUT = Total Phosphorus.

CDUT = Cadmium

The results of the Hillary catchment suggest that the DWF of the combined sewer area was quite typical of municipal sanitary wastewaters. The standard deviation values suggest that the qualities were reasonably stable.

The results of the Black Creek STS were generally comparable to those of the combined sewer area, although the suspended solids, lead and zinc concentrations were somewhat lower and copper concentration was higher than those of the combined sewer area. No explanation can be given for the differences. The results of the Black Creek sanitary sewer area were assumed to be representative of the entire sanitary sewer area in later model simulation work.

As expected, the DWF of the St. Clair Avenue sanitary sewer did show high concentrations of $BOD_5$, suspended solids, total and soluble phosphorus that are normally observed in meat prosessing wastewater. The flow from this sewer, however, is only 1/78 of the flow of the Black Creek STS and, therefore, was not of particular importance to this study.

### 4.4 Precipitation Data

Precipitation data obtained at the Castlefield Works Yard station in April to October, 1983 were segregated into events for further analysis. A storm event was defined as a period of precipitation not separated by a no-precipitation period of longer than 6 hours. The use of the 6-hour period for separating events was arbitrary but it was believed that this duration was reasonable because the impact of a storm on the sewer system was expected to have diminished to a negligible degree after precipitation had ceased for 6 hours. The same period was used for defining storm events in another TAWMS project (Pitt, 1984).

Precipitation results are shown in Table 4.5. There were 58 events in the season. The total seasonal precipitation was 369.8 mm and the average precipitation was 6.4 mm per event. The season in 1983

TABLE 4.5

OBSERVED PRECIPITATION AT CASTLEFIELD WORKS YARD

(A) Summary

| | Observed (1983) | AES 1966-81 Avg. |
|---|---|---|
| Observation Period: | April-October | April-October |
| Total No. of Events: | 58 | 64 |
| Total Precipitation: | 369.8 mm | 447.7 mm |
| Avg. Precipitation per Event: | 6.4 mm | 6.9 mm |

(B) Frequency Distribution of Observed Events

| | Event Precipitation (mm), Not More Than | | | | | | | More Than |
|---|---|---|---|---|---|---|---|---|
| | 2 | 4 | 6 | 8 | 10 | 15 | 20 | 20 |
| No. of Events | 21 | 11 | 6 | 6 | 2 | 5 | 3 | 4 |
| Cumulative No. of Events | 21 | 32 | 38 | 44 | 46 | 51 | 54 | 58 |

Note: AES = Atmospheric Environmental Services of Environment Canada.

was somewhat drier compared with the statistics of the average season between 1966 and 1981 of the Environment Canada precipitation station at Old Weston Road.

## 4.5 Combined Sewage Quantities and Qualities

Combined sewage quantity data were collected in 14 storm events in the Hillary catchment. This section provides a general appraisal of the data. The use of the data for model simulation will be discussed in the appropriate sections later.

Precipitation in the storms ranged from 4 mm to 30 mm (Table 4.6). Inflow to the Hyde Avenue tank occurred in each of the 14 events but the tank was filled only in 9 events, most of which had a precipitation more than 12 mm. Site 3 overflowed in each of the 14 events. Hydrographs of these events are in Appendix B2 and an example is shown in Figure 4.1. One obvious conclusion from the example is that reduction of DWF will not be effective in reducing CSO since the DWF rate is only a small fraction of the combined sewage flow rate. On the other hand, reduction in stormwater runoff, if it is feasible, will effectively reduce the combined sewage flow rate.

Observed combined sewage suspended solids (SS) concentrations are plotted against observed flow rates in Figure 4.2. Detailed data are in Appendix B2. SS are of particular interest because the concentrations of several of the other pollutants could be related to SS concentrations as will be demonstrated later.

The SS data appear reasonably normal except possibly in two places. First, the data of the beginning part of the event on August 8, 1983 were erroneous because the Black Creek momentarily flooded the overflow weir of the Hyde Avenue tank. The erroneous data points were not used. Second, there was a sudden change from a low SS concentration to a high SS concentration in the last two points on the curve of the event on September 16, 1983. A trend like this was not observed in the other events. The last point on the curve was

TABLE 4.6

LIST OF MONITORED EVENTS IN HILLARY CATCHMENT

| Date | Event Precip. (mm) | Use of Data: Model Calib. | Use of Data: Model Verif. | Use of Data: Load Rate | Occurrence of Phenomenon: Inflow to Tank | Occurrence of Phenomenon: Overflow From Tank | Occurrence of Phenomenon: Overflow at Site 3 | Comments |
|---|---|---|---|---|---|---|---|---|
| 30 Apr 83 | 19.3 | Y | | | Y | Y | Y | |
| 1 May 83 | 7.9 | Y | | | Y | | Y | |
| 2 May 83 | 14.0 | Y | | | Y | Y | Y | |
| 19 May 83 | 21.9 | Y | | Y | Y | Y | Y | |
| 22 May 83 | 11.6 | Y | | | Y | | Y | |
| 29 May 83 | 11.6 | | Y | Y | Y | | Y | |
| 6 June 83 | 6.7 | Y | | Y | Y | | Y | |
| 4 July 83 | 4.3 | | Y | Y | Y | | Y | |
| 8 Aug 83 | 24.7 | | | Y | Y | Y | Y | Part of Event. |
| 22 Aug 83 | 20.2 | | Y | Y | Y | Y | Y | |
| 16 Sep 83 | 29.5 | | | Y | Y | Y | Y | Flow data incomplete. |
| 21 Sep 83 | 7.4 | Y | | | Y | Y | Y | |
| 12 Oct 83 | 19.0 | | | Y | Y | Y | Y | Precip. gauge problem. No SS data. |
| 13 Oct 83 | 14.2 | Y | | Y | Y | Y | Y | |

Note: Y = Yes

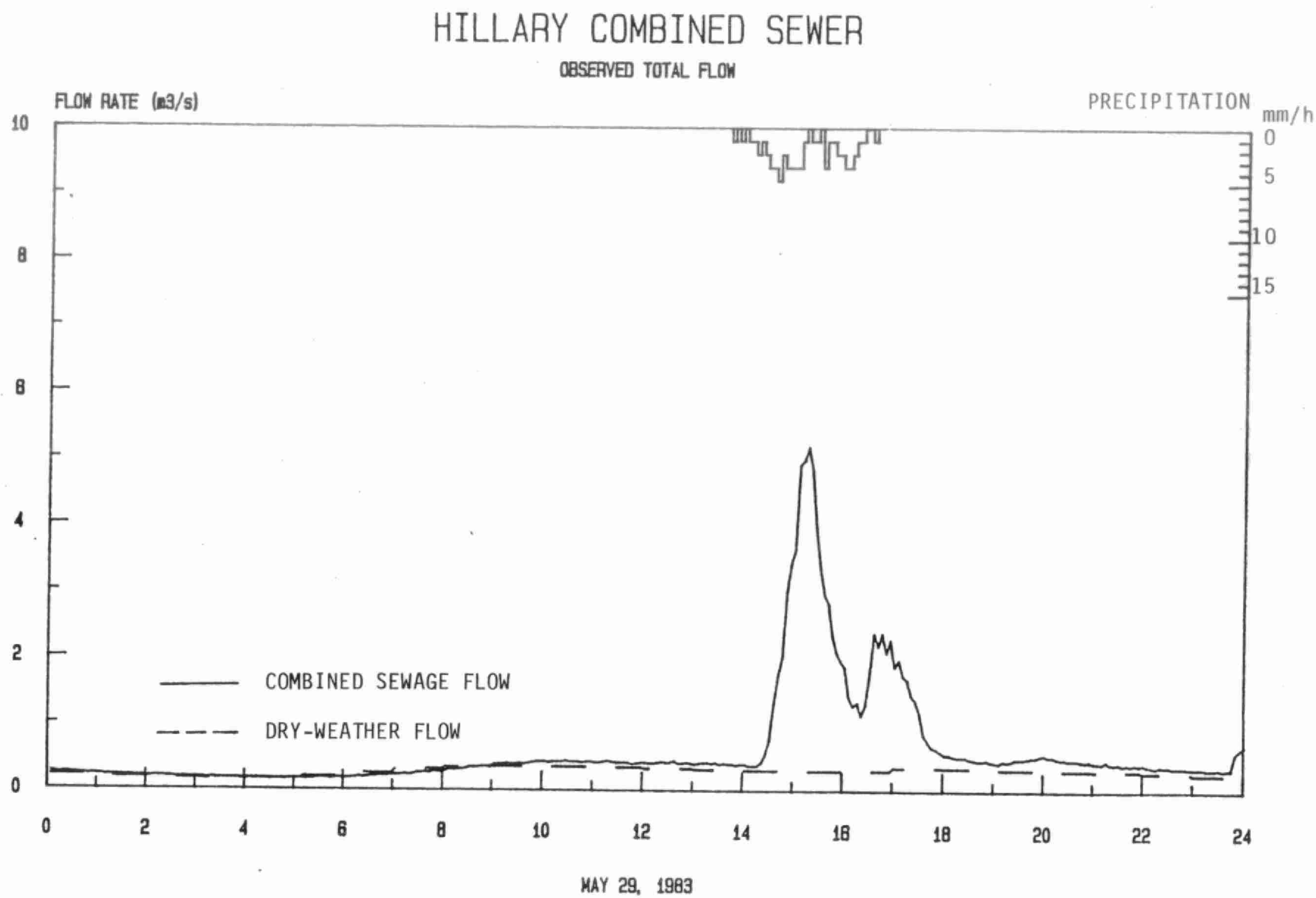



FIGURE 4.1 : COMBINED SEWAGE HYDROGRAPH

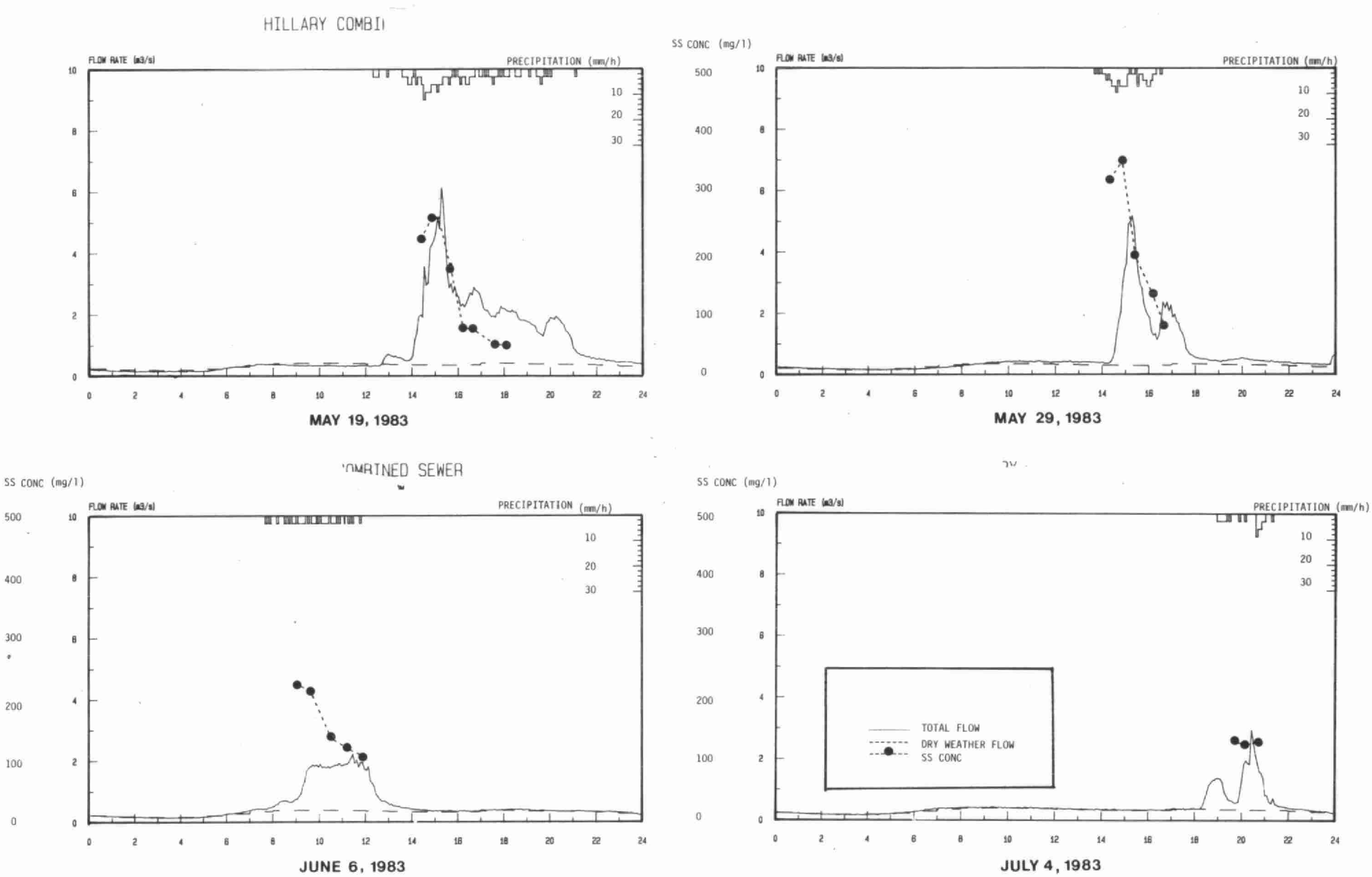



**FIGURE 4.2 : COMBINED SEWAGE SUSPENDED SOLIDS CONCENTRATIONS (sheet 1 of 2)**

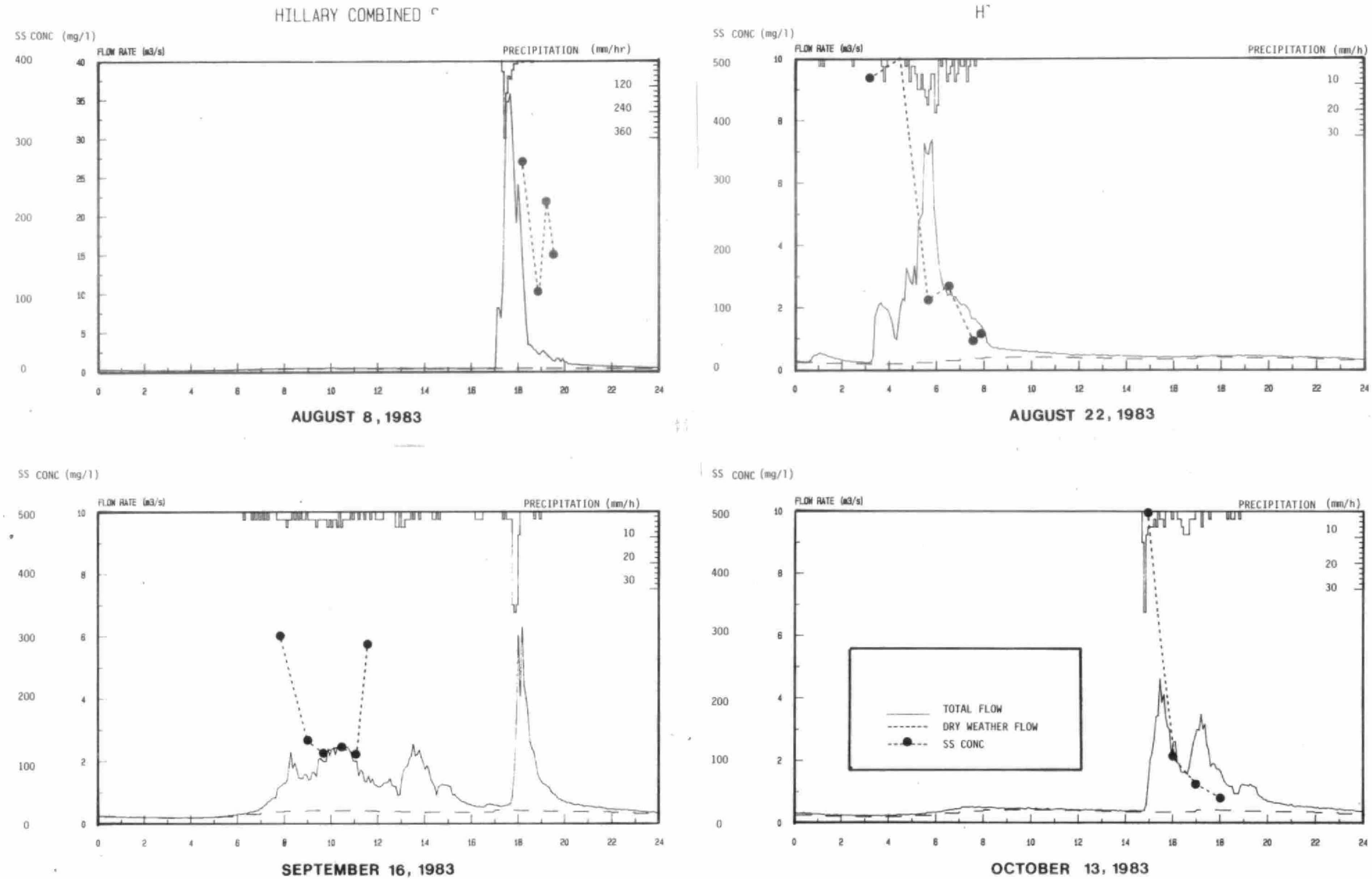



**FIGURE 4.2 : COMBINED SEWAGE SUSPENDED SOLIDS CONCENTRATIONS (sheet 2 of 2)**

apparently a random error which may be expected according to theories of statistics. The cause of the error could not be traced. Since this point lies well within reasonable bounds of the entire set of SS concentrations (the point is about 1 standard deviation from the mean concentration of the entire set), the point was not rejected.

It is interesting to note that the SS concentrations varied approximately proportionally with the flow rates. This relationship provided the basis for relating the SS concentration to the flow rate in the model simulation discussed later.

The flow-weighted concentrations of SS, $BOD_5$, total phosphorus and fecal coliforms of the observed events are summarized in Table 4.7. The results appear to be comparable to literature values as illustrated in Table 4.8. The comparison should not be made too strictly, of course, because combined sewage is variable in nature and the catchments are not identical.

## 4.6 Wet Weather Inflow/Infiltration of Sanitary Sewer Area

There were indications that the wet-weather inflow/infiltration (I/I) in the sanitary sewer area was notable in certain locations but negligible in some others (City of Etobicoke, 1983). However, no field studies had been carried out by municipalities to estimate the magnitudes and distribution of wet-weather I/I in the sanitary sewer area. Resources and time allotment for this CSO study precluded the undertaking of a field study for this purpose either.

In this study, the wet-weather I/I in the sanitarty sewer area was estimated from wet-weather daily flow volumes recorded on the Humber WPCP log sheets. Data of year 1980 were used since they gave the best correlation with the precipitation of the year. The following relationship was derived:

TABLE 4.7

SUMMARY OF OBSERVED COMBINED SEWAGE QUALITY DATA

| | Number of Events Sampled | Total No. of Samples | Flow-weighted Avg. Conc. (mg/l) | Coef. of Variation |
|---|---|---|---|---|
| Suspended Solids | 8 | 42 | 196. | 0.68 |
| BOD5 | 9 | 48 | 55. | 0.61 |
| Total Phosphorus | 9 | 44 | 1.96 | 0.35 |
| Filtered Phosphorus | 9 | 48 | 0.44 | 0.73 |
| Cadmium | 7 | 38 | 0.006 | 0.83 |
| Copper | 7 | 38 | 0.119 | 0.59 |
| Lead | 7 | 38 | 0.182 | 0.61 |
| Zinc | 7 | 38 | 0.300 | 0.62 |
| Fecal Coliforms | 14 | 81 | 6.218* | 0.11 |

* Log mean, No./100ml

TABLE 4.8

COMPARISON OF COMBINED SEWAGE CONCENTRATIONS

| Location | Average Concentrations (mg/l) (1) Suspended Solids | BOD5 | Phosphorus | F.Coliforms No./100 ml | Site Descriptions & Remarks |
|---|---|---|---|---|---|
| Hillary Catchment | 196 | 55 | 1.96 | | Study site. |
| Belleville WPCP Overflow (2) | | | | | Mainly resid./com. Total city 2,350 ha. CSO area 50 ha. |
| Events with "1st Flush" | 522 | 118 | 5.3 | - | 9 events with "1st flush" |
| No "1st Flush" | 191 | 68 | 4.0 | | 5 events no "flush" |
| New Haven CSO Area (3) | 226 | 302 | - | - | Residential. 6 ha. 2 events observed. |
| Milwaukee CSO Area (4) | 361 | 87 | - | Range 15-12,000 | Area 2,428 ha. 70% resid.,18% com., 11% ind. 15 events for SS,12 for BOD, 3 for FC. |
| Cleveland CSO Area (5) | 234 | 92 | - | 4.5x10^6 | Area 24,800 ha. Population 600,000. Resid/com/ind. With some large industries. Data of 193 to 197 observations. |

Notes:
(1) Except Fecal Coliforms
(2) Kronis, H. , 1975
(3) Cermola, J.A. et al, 1979
(4) Meinholz, T.L. et al, 1979
(5) Nebolsine, R. et al, 1972

$$II = 1.475 \times P^{1.446}$$

where $II$ = I/I in a storm event in 1,000 $m^3$

$P$ = Precipitation in event in mm.

Coefficient of correlation is 0.66

This relationship was used later to predict the I/I volume of a given storm event in model simulation. The I/I volume was distributed over the storm event duration, using a unit hydrograph which was derived from the Humber WPCP hourly flow volumes in the summer of 1983. Figure 4.3 is an example of the I/I hydrograph so generated. Details of the methodology for computing the I/I are in Appendix B3.

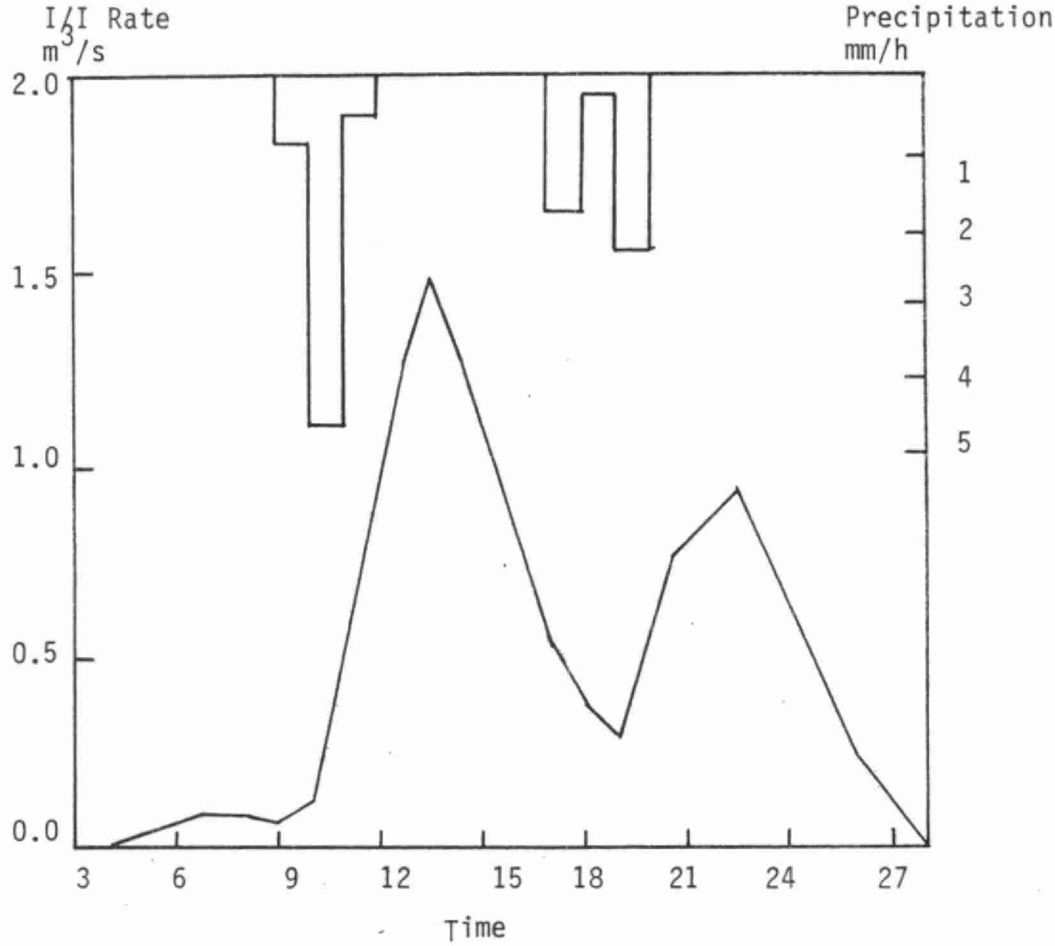



FIGURE 4.3: ILLUSTRATION OF INFLOW/INFILTRATION HYDROGRAPH
(JULY 10, 1979)

# 5.0 MODEL SIMULATION

## 5.1 Need for Judicious Use of Model

Section 5.0 presents briefly a general concept of computer model simulation and describes the setting up of the model for this CSO study. Applications of the model will be discussed in Sections 6.0 and 7.0.

In general, it is not practical to obtain CSO statistics for a complete, typical hydrologic season by field studies alone. Nor is it possible to forecast from observed data the changes in the statistics resulted from CSO control. The missing information needs to be obtained by model simulation as was done in this study. The role of data collection presented earlier was to provide an understanding of the responses of the study system to hydrologic changes and to obtain data required for using the model.

Model simulation was already used in engineering practice in the age of hand calculation. The advent of the computer makes it possible to calculate much faster and to do complex mathematics with fewer simplifying assumptions. The computer also makes it possible to store and print voluminous information at great speed and low cost and this ability meets the needs of input/output requirements of model simulation just well. In these ways, the computer augments the capability of model simulation, but the computer is no substitute for the knowledge of and the experience in the application of engineering principles in model simulation. The need for applying computer model simulation with caution may be reflected by the following quotation:

> "However, in a major model, there are many relationships which are incorporated without being unequivocably assessed to be true; this important aspect seems to have escaped major criticism..... computer approaches not based upon proved scientific relationships can be highly misleading." (Whipple, 1977).

Even if the theories used in a model are sound and the modelling is done properly, the need for intelligent appraisal of modelling results cannot be over-emphasized. This is true of any urban environmental engineering model, even if it is a state-of-the-art model. The attainable degree of accuracy of modelling varies according to the type of simulation. For sewer flow calculations, the accuracy is usually high; for estimation of stormwater runoff, the accuracy is reasonable; and for estimation of pollutant loadings from stormwater runoff (and combined sewage), the accuracy is comparatively lower. The natural processes that influence the generation and transport of pollutants are not yet fully identified or understood. No mathematical formulas can incorporate all the influencing factors of the processes. Even if the formulas could do so, it is not practical to collect all the required input data and to find a suitable computer system to do the computation. Consequently, many simplifying assumptions are still needed in pollutant loading modelling. The above discussion is not to discredit the usefulness of modelling, but is an acknowledgement of the need to interpret modelling results intelligently in the light of the limitations of this technology, which is the best available today.

It is felt more appropriate to interpret pollutant modelling results in the context of long-term statistics than on a single event basis, and in the context of noting the changes in results as simulated conditions change than taking the numerical results too dogmatically.

## 5.2 Selection of Model

Many simulation models exist in the private market and public domain. Only those models that had received some formal evaluation by a reputable agency were considered for this study, as it was beyond the study scope to evaluate models and it was unwise to use a model that had not been assessed critically.

Eighteen major models had been assessed by the U.S. Environmental Protection Agency (Brandstetter, 1976). Four of the models, namely,

Battelle Urban Wastewater Management Model, Dorsch Quantity/Quality Simulation Model, EPA Stormwater Management Model, and Hydrocomp Simulation Program, met the basic requirements of this CSO study with respect to catchment hydrology, sewer hydraulics and wastewater quality simulation. The first two named models are proprietary. The Hydrocomp model requires extensive resources support that was neither justifiable nor available to this study. The U.S. EPA Stormwater Management Model (latest SWMM Version III.3) was used. In any case, SWMM is the most widely tested and used model both in research and engineering practice for this type of studies.

### 5.3 Description of the Study Model

The SWMM model consists of 6 blocks each being capable of performing specific functions. Two of the blocks, the Executive and the Runoff, are most often used in most projects using SWMM. In this CSO study, three blocks, the Executive, the Runoff and the Transport were used.

The Executive block is a program manager. The Runoff block is responsible for all computations relating to runoff quantities and qualities and their routing through catchments. The Transport block merges input DWF and infiltration quantities and qualities with runoff results to simulate combined sewage and then routes the combined sewage through the sewer system. Combined sewer overflow is simulated by this block. The algorithms used by the model are discussed at length in the user manual of the model (Huber, 1981).

The SWMM model was set up, or "personalised", to represent the study system by inputting catchment and sewer data and by providing coefficient values of certain mathematical functions to the model to enable the model to estimate flow quantities and pollutant concentrations. The 8 pollutants listed in the study scope were studied.

The following paragraphs outline briefly the model that was set up. Further details are in Appendix C1.

## Sewers and Humber WPCP

Trunk sewers downstream of the regulators and the regulators themselves were included in the model. The WPCP was represented by the limiting capacity of 11.8 $m^3/s$. An assumption made was that any primary effluent in excess of the secondary treatment capacity of 9.6 $m^3/s$ would bypass the secondary treatment process to the effluent outfall after chlorination. Figure 5.1 is a logic layout of the sewer system simulated.

## Flow Quantity Calculation

The combined sewer area was represented in the model by 26 subcatchments. The input data included sub-catchment size, percent of impervious surface, ground slope, catchment width, soil moisture coefficients, depression storage and the "family tree" of the sub-catchments for flow routing. Runoff quantity calculation involved the application of the kinematic wave theory, Horton soil moisture equation, and the linear reservoir theory. The calculation process was able to account for the effects of catchment topography; loss of precipitation through infiltration and depression storage; recovery of infiltration capacity and depression storage in a dry period; and change in precipitation intensity.

DWF quantities and qualities of both the combined sewer area and the sanitary sewer area were input, as was wet-weather I/I of the sanitary sewer area. Wet-weather I/I of the combined sewer area was part of the combined sewage.

## Runoff Quality Calculation

Input data for runoff quality prediction were derived from the observed combined sewage data. The derivation is in Appendix C2. The following describes briefly the methodology for runoff quality prediction.

Washoff of SS and $BOD_5$ by stormwater runoff was based on the rating curves shown in Figures 5.2 and 5.3 respectively. The

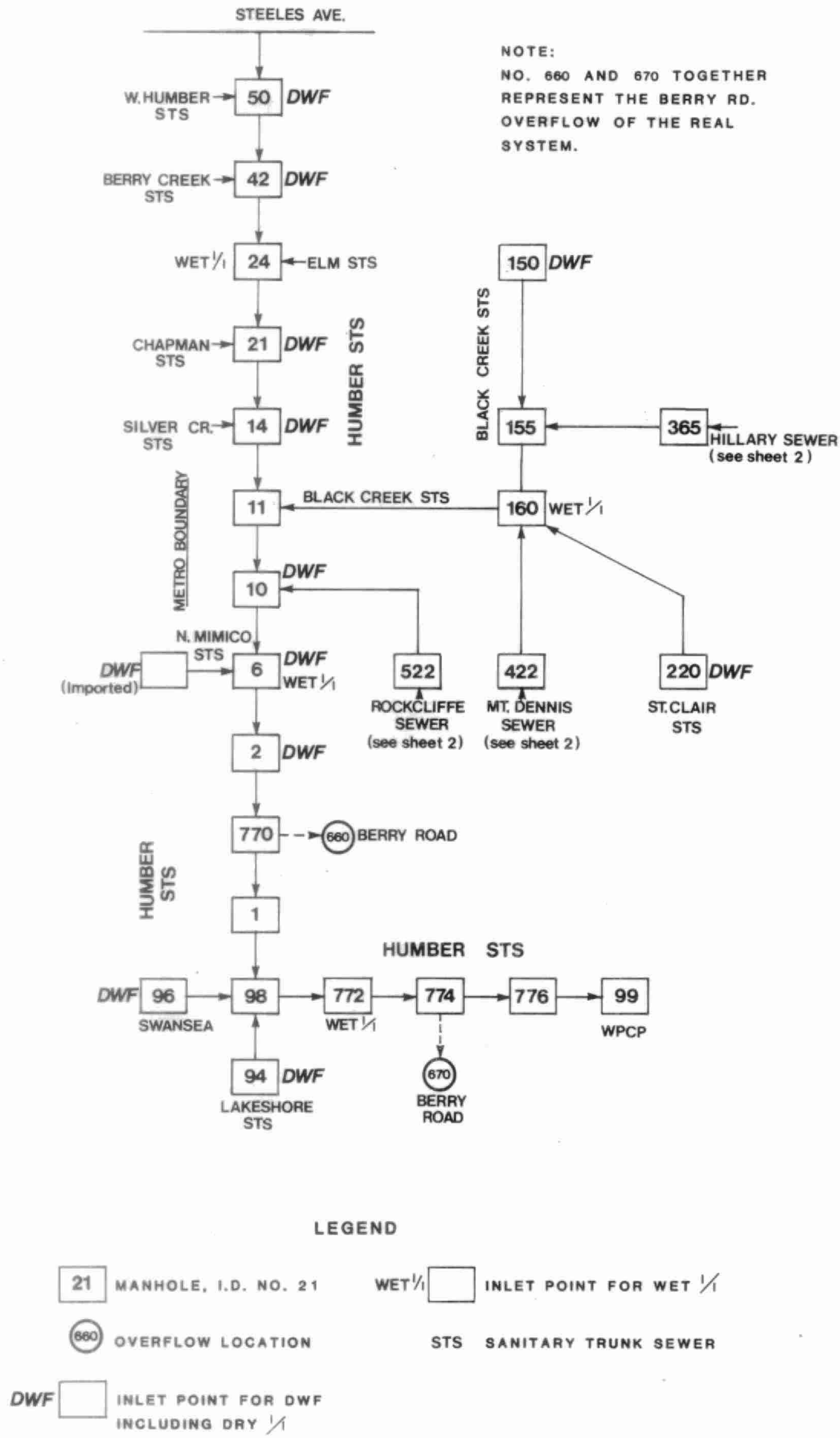



**FIGURE 5.1: STYLIZED LOGIC DIAGRAM OF MODELLED SYSTEM**
**( sheet 1 of 2 )**

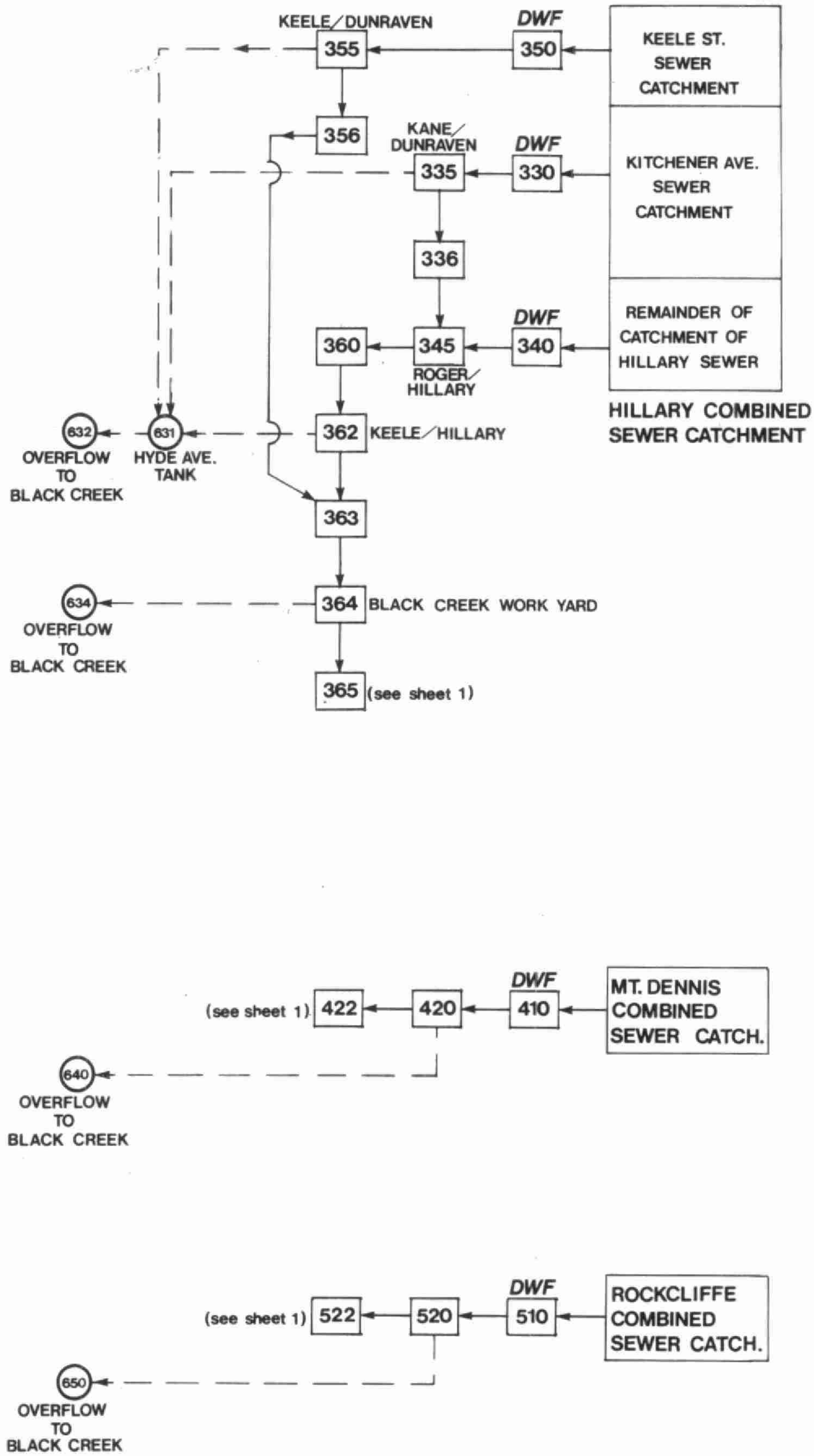



**FIGURE 5.1: STYLIZED LOGIC DIAGRAM OF MODELLED SYSTEM**

**( sheet 2 of 2 )**

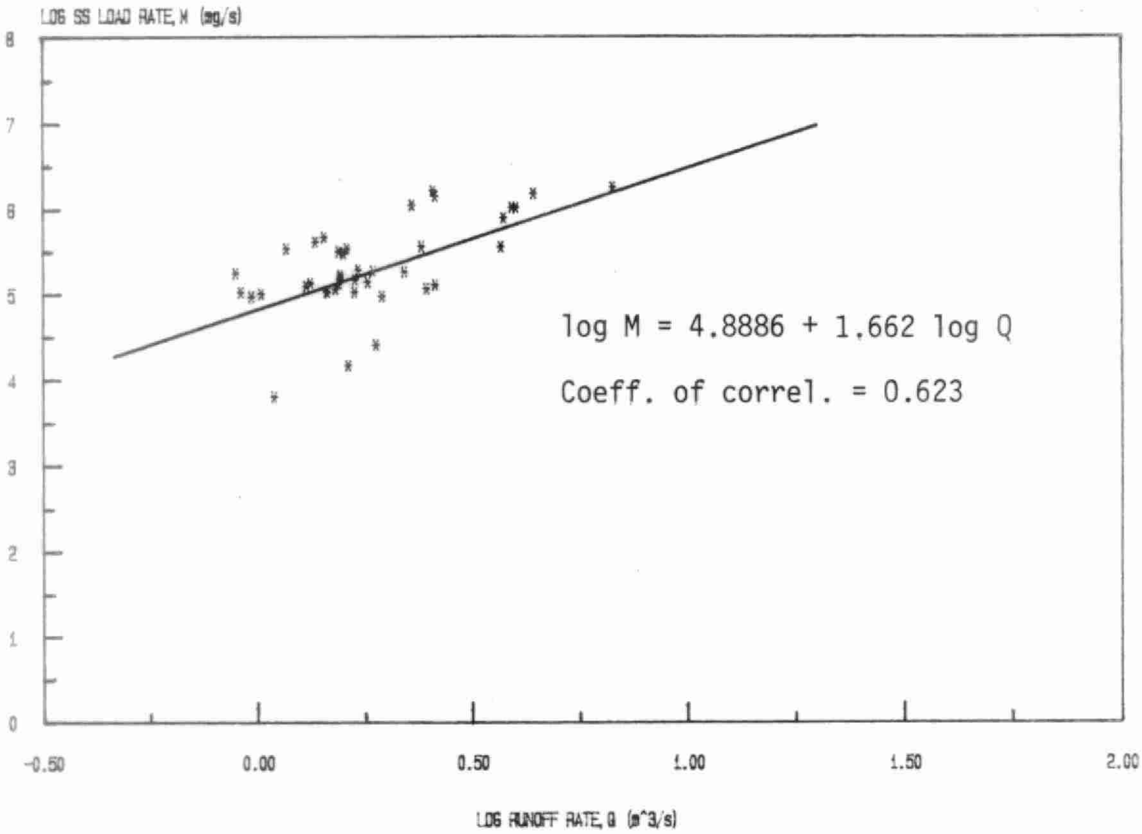



FIGURE 5.2 : SUSPENDED SOLIDS RATING CURVE

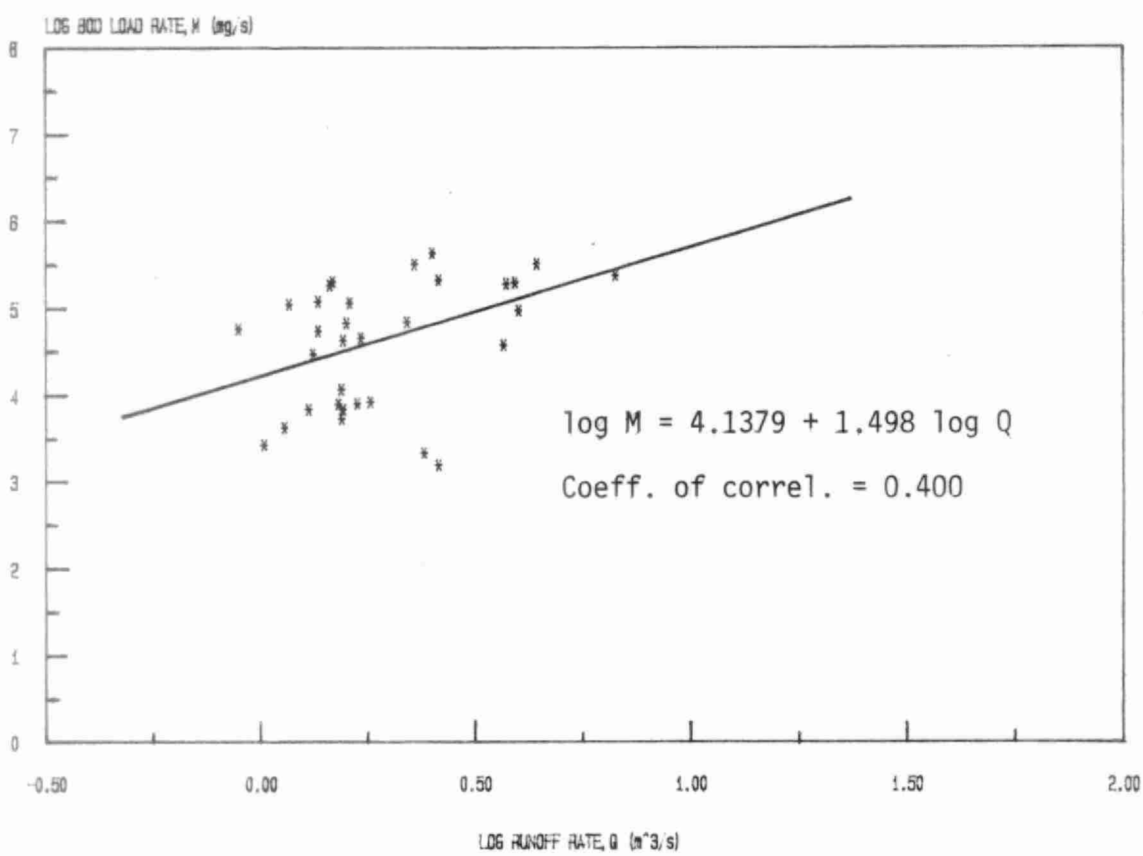



FIGURE 5.3 : $BOD_5$ RATING CURVE

coefficient of correlation (0.4) for the $BOD_5$ curve is low. However, since $BOD_5$ is not a pollutant of great concern to the Humber river, no attempt was made to obtain additional data to produce a better curve.

Washoff of the four heavy metals and total phosphorus was related to SS washoff in the following ratios:

| | Total Phosphorus | Cadmium | Copper | Lead | Zinc |
|---|---|---|---|---|---|
| Ratio of Pollutant to SS (mg/gm) | 9.17 | 0.038 | 0.645 | 1.125 | 1.886 |
| Coeff. of Variation | 51% | 110% | 65% | 60% | 48% |

The derived runoff concentrations of soluble phosphorus indicated that they were not correlated with the runoff rates. As a result, the derived averaged concentration of 0.54 mg/l was used.

CSO fecal coliform loading was estimated by using the observed combined sewage geometric mean concentration of $1.65 \times 10^6$ counts/100 ml. It was believed that there was yet no credible method and data for forecasting the rate of fecal coliform production on a catchment or the rate of washoff from the catchment. Fecal coliform concentrations are influenced by ambient temperature, light intensity, nutrient conditions and possibly some other biological factors but it is yet not possible to identify and quantify all the major influences. Recent Ontario studies (Gore & Storrie, 1981; Gore & Storrie, 1984) indicated the difficulty even to completely identify the sources of observed fecal coliform loadings. Therefore, although superficially sophisticated methods exist for forecasting fecal coliform washoff rates, the use of the observed mean concentration was preferred.

The amount of pollutant washoff was limited by the amount of the pollutant accumulated on the catchment surface. SS accumulation rates were based on Figure 5.4. Limiting accumulation of $BOD_5$, total phosphorus and the four heavy metals was related to the SS accumulation curve by the ratios mentioned earlier. As soluble

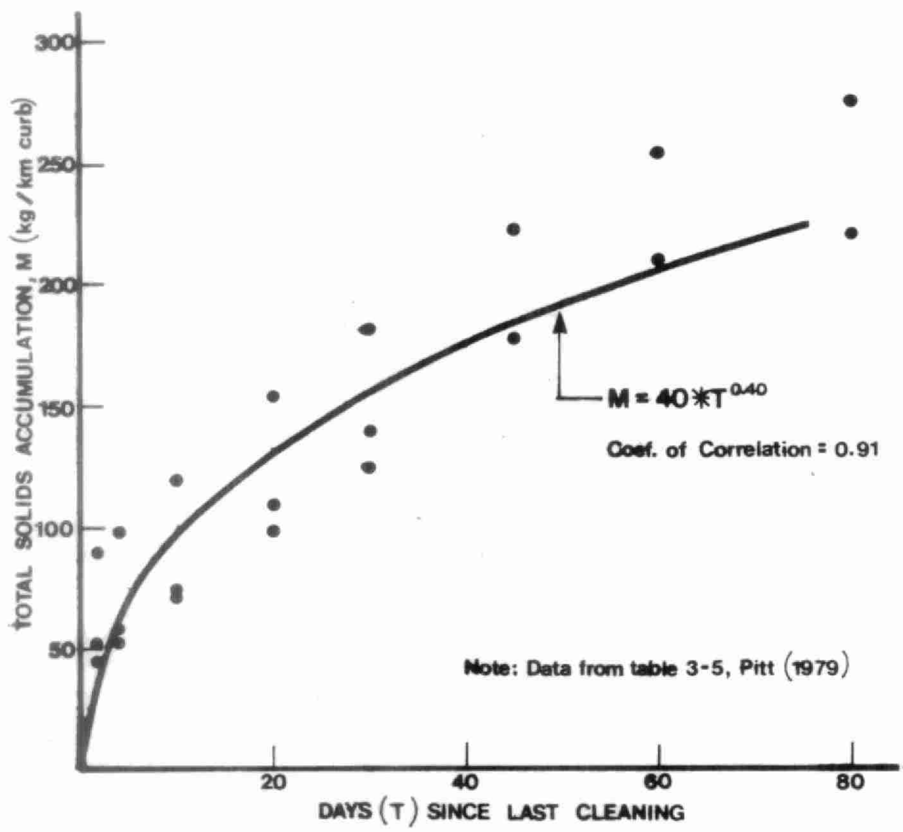



FIGURE 5.4: TOTAL SOLIDS ACCUMULATION

phosphorus concentration was not correlated with stormwater runoff, it was inferred that it was also not related to accumulation. Fecal coliforms were assumed to be plentiful on the surfaces of catchments and would not be depleted by stormwater washoff.

## 5.4 Model Calibration: The Prevailing Practice

In principle, after a model is set up and before it is commissioned, it should be test-run (calibrated) to check if predicted results for selected storm events agree reasonably with observed data of the same events and values of model parameters should be adjusted as necessary. In practice, however, a model is often applied without calibration (NCASI, 1982).

There is yet no standardized practice in model calibration. A few examples will illustrate the variety of approaches used. In a combined sewer study in New Haven using SWMM (Cermola, 1979), one storm event each was used for calibration and verification. In a Hamilton combined sewer study using the Runoff block of SWMM (James, 1980), 4,000 ha out of a 6,800 ha study area were calibrated for flow quantity. In an Atlanta stormwater management study (Holbrook, 1976), the observed flow quality data used consisted of grab samples and composite samples taken from the beginning of overflow to a point "well beyond the first flush".

The goodness of calibration is often judged by comparing the observed and predicted hydrographs visually. Some recent studies compared the sum of observed and predicted values (Gore & Storrie, 1980; Marshall, Macklin, Monaghan, 1982). The use of statistical measures to judge the goodness is not common but it has been advocated as a more scientific approach (NCASI, 1982).

## 5.5 Flow Quantity Calibration

In this CSO study, flow quantity calibration was carried out with the Hillary catchment which makes up 72% of the total combined sewer area. Eight of the observed storms in 1983 (Table 4.6) were used for calibration. They were chosen to represent three ranges of

precipitation: below 10 mm per event; between 10 and 20 mm; and over 20 mm. For precipitation, data observed at the Castlefield Works Yard were used.

The "goodness" of calibration was measured by the linear regression equation of predicted event runoff volumes vs observed event volumes and the following three criteria for the measurement were proposed:

(1) The slope of the regression line to be within 0.9 and 1.1.

(2) The intercept of the regression line with the horizontal or vertical axis to be within - 2,690 $m^3$ and + 2,690 $m^3$.

(3) The coefficient of correlation of the regression line to be within 0.9 and 1.0.

The meaning of the criteria is illustrated in Figure 5.5. The ideal case, Figure 5.5(A), represents the ideal but unattainable situation in which every predicted event volume agrees exactly with the observed volume of the corresponding event. The slope of the line is 1.0, the intercept is 0 and the coefficient of correlation is 1.0. The regression line is a measure to see how far the actual case deviates from the ideal case.

If the actual regression line tilts upwards from the ideal line, as in Figure 5.5(B), the model will err on the high side and the absolute magnitude of the error increases with larger events. The error will be on the low side if the line tilts downwards.

If the actual regression line is parallel to the ideal line, but intercepts an axis at, say, 2,000 $m^3$ as illustrated in Figure 5.5(C), the predicted volume is 2,000 $m^3$ smaller than the true volume. The intercept represents a constant error no matter what the size of the event is.

Hypothetically, the ideal line could be obtained with very scattered points as illustrated in Figure 5.5(D), but the coefficient of correlation will be much smaller than 1. If the coefficient of

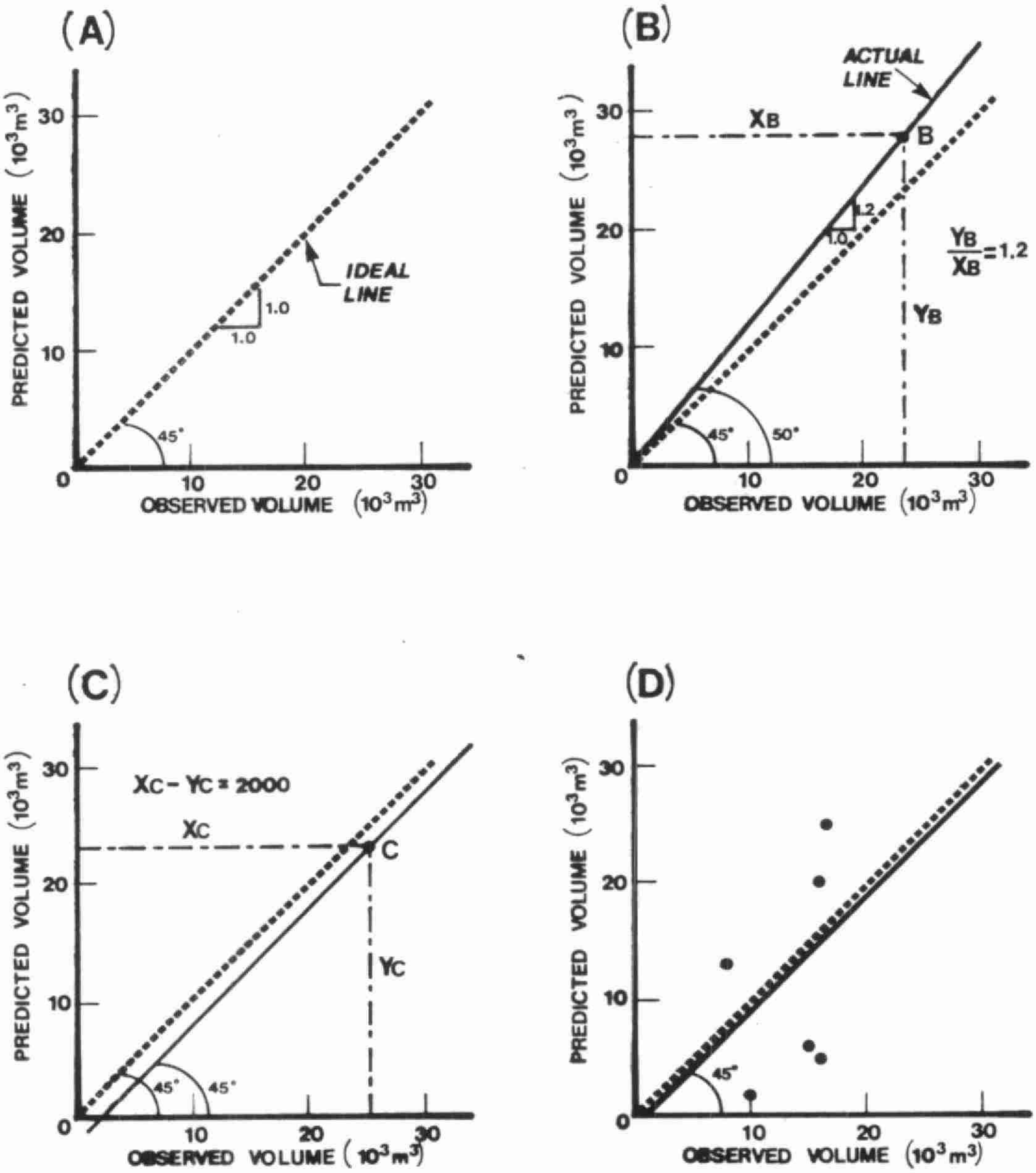



FIGURE 5.5: MEANING OF CALIBRATION CRITERIA

correlation is small, the chance of having a point falling exactly on the line is not high.

The proposed criteria can now be interpreted in the light of the foregoing discussions. Criterion (1) restricted the variable error to 10% of the true event volume. Criterion (2) restricted the constant error to 2,690 $m^3$ which was 10% of the average observed DWF of the Hillary catchment. Criterion (3) was to ensure that there would be a reasonable chance for the relation between observed and predicted volumes to be truly represented by the regression line.

The criteria were arbitrary, but it will be realized that they were fairly stringent for stormwater modelling.

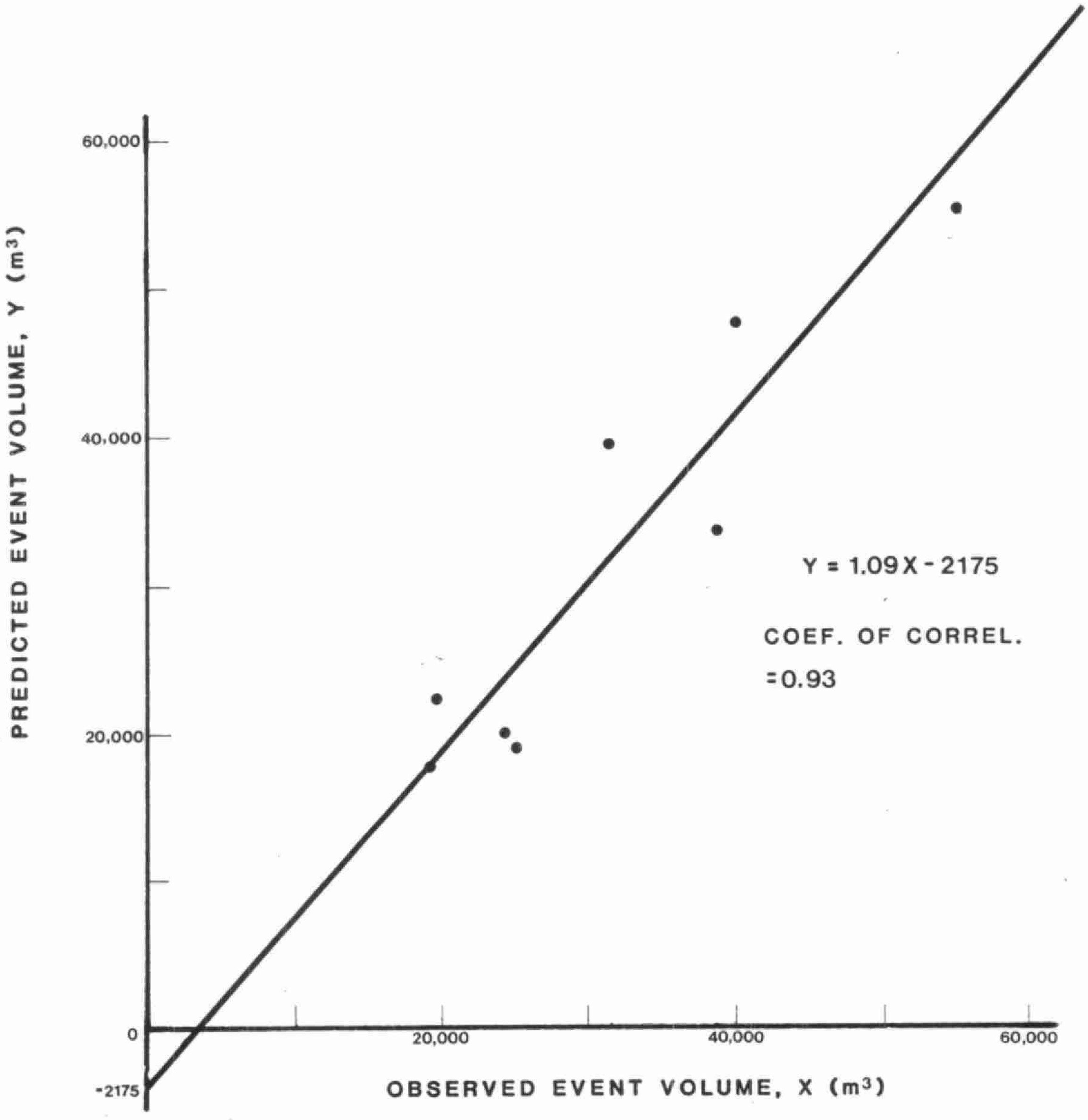
The actual calibration results are shown in Figure 5.6. As all the three criteria were satisfied, the model was considered calibrated. The hydrographs of the calibration events are in Appendix C3.

The model was verified with 3 other observed events. The results are shown in Table 5.1. It can be shown that there is no difference between the predicted and observed results at 90% confidence level according to a student t statistical test (Mendenhall, 1978). So the model was verified.

## 5.6 Flow Quality Calibration

Flow quality calibration of the study model differed from flow quantity calibration in that each coefficient used in the mathematical expressions for computing accumulation and washoff of each pollutant was derived from observed data and then input to the model. Therefore, the input data in effect calibrated the model. This approach was also used elsewhere (Marshall, Macklin, Monaghan, 1982).

To illustrate that the model was indeed calibrated for predicting flow qualities, the predicted and derived SS concentrations (Table 5.2) were put to the student t test (Mendenhall, 1978). It can be shown that there was no difference between the two sets of data at 90% confidence level.




| EVENT | 830430 | 830501 | 830502 | 830519 | 830522 | 830606 | 830921 | 831013 | |
|---|---|---|---|---|---|---|---|---|---|
| X | 39824 | 25076 | 38820 | 54632 | 19469 | 19280 | 24298 | 31666 | SUM = 253065 |
| Y | 48335 | 19308 | 33700 | 55666 | 22300 | 18172 | 21570 | 39436 | SUM = 258487 |

FIGURE 5.6 : CALIBRATION OF FLOW QUANTITIES

TABLE 5.1

QUANTITY VERIFICATION OF MODEL

| Event Date | Predicted Event Volume (m^3) | Observed Event Volume (m^3) |
|---|---|---|
| 830529 | 26,409 | 21,653 |
| 830704 | 9,265 | 8,363 |
| 830822 | 48,082 | 43,996 |

TABLE 5.2

QUALITY VERIFICATION OF MODEL (SUSPENDED SOLIDS)

| Event Date | Predicted Runoff Flow-wt Conc. (mg/l) | Observed Runoff Flow-wt Conc. (mg/l) |
|---|---|---|
| 830519 | 137 | 151 |
| 830529 | 146 | 255 |
| 830606 | 88 | 133 |
| 830704 | 93 | 107 |
| 830822 | 185 | 269 |
| 830916 | 214 | 122 |
| 831013 | 215 | 157 |

## 5.7 Selection of Simulation Period

In practice, when a model as complex as SWMM is used for planning studies, simulation is run often for a period of not more than one year (Marshall, Macklin, Monaghan, 1982; Dorsch Consult Ltd, 1979). In the present CSO study, simulation was done for a selected season (April to October) of a selected year (1979). The rationale for selecting the simulation period is explained below.

Typically, CSO is most pronounced in the summer, because summer storms have high intensities. On the average, a scheme designed for CSO control in the summer (April to October) will be able to control CSO in the remainder (November to March) of the year. An earlier CSO modelling study (Dorsch Consult Ltd., 1979) also made use of this seasonal characteristic to simulate for the April-October season only.

Second, urban hydrology of frozen ground and snow-melt is still in the developmental state and few data are available (Waller, 1974(?)). A Hamilton CSO study using SWMM (Robinson, 1981) also cited this technical limitation as a reason for simulating for the April-October season only.

Finally, historical precipitation data needed for model input were collected for the April-October season only by the Atmospheric Environmental Services (AES) of Environment Canada.

The selection of year 1979 was based on precipitation statistics.

Three AES precipitation stations (Toronto Downsview, Toronto Old Weston Road and Toronto Etobicoke) closest to the combined sewer area were considered to determine how the precipitation recorded at the stations should be weighted to allow for uneven distribution of precipitation over the combined sewer area. Based on the Thiessen method (Linsley, 1975), it was concluded that precipitation in the combined sewer area should be represented solely by the Old Weston Road station data.

The Old Weston Road station had complete data for the April-October season for 16 years from 1966 to 1981, recorded at hourly intervals. A computer program PRCPSTAT was developed in this study to perform the statistical analysis. The precipitation statistics of the April-October season of 1979 (Table 5.3) compared most closely to the average statistics of the 16-year period, so year 1979 was selected.

Huge efforts would be required to do simulation for each of the 64 storm events in the season, because the Transport block of SWMM can handle one event only in each run of the computer program. As an alternative, as shown in Table 5.4, each event with precipitation more than 10 mm and one event each from the next three lower ranges of precipitation were simulated. To produce statistics for the season, the simulation results for the groups of storms with 8-10 mm, 6-8 mm, and 4-6 mm of precipitation were multiplied by the group factors 5, 3 and 5 respectively. Therefore, 27 events were in effect simulated.

The remaining 40 storm events in the ranges of not greater than 4 mm of precipitation were ignored. They yielded a total precipitation of 54 mm which was 13 % of the season's total. The total runoff produced by these minor storm events was expected to be less than 13% because minor events had proportionally higher precipitation loss to evaporation and so forth than larger events. Available observed data (Table 5.5) indicated that overflow in minor storms (not more than 4 mm precipitation) was mostly marginal. It was expected that CSO in minor storm events would be readily eliminated with the minimum CSO control. Ignoring the minor storm events actually had the advantage of eliminating distortion of CSO frequency statistics.

With completion of the pre-requisites, the study model was ready for application.

TABLE 5.3

PRECIPITATION STATISTICS OF AES OLD WESTON ROAD STATION

| | Seasonal Average of 1966-1981 Period | | | | 1979 Season | | |
|---|---|---|---|---|---|---|---|
| | Per Event | | Per Season | | Per Event | | |
| | Mean | Std Dev | Mean | Std Dev | Mean | Std Dev | Seasonal Total |
| Number of Events | | | 64 | | | | 68 |
| Precip. Duration (hr) | 6 | 7.5 | 410 | 68.7 | 6 | 7.2 | 463 |
| Precip. Depth (mm) | 6.9 | 9.74 | 447.7 | 81.19 | 6.3 | 8.23 | 434.5 |
| Avg. Intensity (mm/hr) | 1.1 | 1.71 | | | 1 | 1.26 | |
| Ante. Dry Duration (hr) | 72 | 84.3 | 4620 | 106.1 | 67 | 72.4 | |

Note : Season is from April 1 to October 31.

TABLE 5.4

SELECTION OF STORM EVENTS FOR SWMM APPLICATION

| Event Precipitation | Number of Occurrences Of Events | Number of Events Selected |
|---|---|---|
| (1) Greater than 10 mm | 15 | 15* |
| (2) Above 8, not exceeding 10 mm | 5 | 1** |
| (3) Above 6, not exceeding 8 mm | 3 | 1** |
| (4) Above 4, not exceeding 6 mm | 5 | 1** |
| (5) Above 2, not exceeding 4 mm | 12 | 0 |
| (6) Not exceeding 2 mm | 28 | 0 |
| Total | 68 | 18 |

Notes:

* Two events in this group occurred on the same day. The two events were treated as one in SWMM simulation.

** Multiply simulation results of event classes (2), (3) and (4) by factors 5, 3 and 5 respectively to calculate seasonal statistics.

TABLE 5.5

OBSERVED OVERFLOW DATA FOR MINOR STORMS (1)

| Date | Precip(mm) | Site 3 Peak Overflow Rate(m3/s) | Mt. Dennis Peak Overflow Rate(m3/s) | Rockcliffe Peak Overflow Rate(m3/s) |
|---|---|---|---|---|
| 83/04/11 | 3.2 | | | |
| 83/04/21 | .3 | | | |
| 83/04/27 | .7 | | | |
| 83/05/04 | 3.7 | .2 | .08 | |
| 83/05/08 | .3 | .021 | .033 | |
| 83/06/03 | 3.2 | .119 | .012 | .111 |
| 83/06/05 | 1 | | | .038 |
| 83/06/30 | 1.2 | | | .023 |
| 83/07/21 | 2 | | | .085 |
| 83/07/28 | 2.7 | | | .525 |
| 83/07/29 | .5 | .053 | | |
| 83/07/31 | 3 | .172 | | 1.286 |
| 83/08/01 | 1 | | | |
| 83/08/03 | 3 | | | |
| 83/08/05 | 1.7 | | .001 | |
| 83/08/27 | 2.8 | .545 | .104 | .658 |
| 83/08/30 | 3.7 | .2 | .033 | .122 |
| 83/09/06 | .5 | | | .019 |
| 83/09/09 | .3 | | | |
| 83/09/20 | 1.5 | | | .111 |
| 83/09/22 | .5 | | | |
| 83/09/23 | 1.7 | | | |
| 83/09/25 | 1.9 | | | |
| 83/10/03 | 2.2 | | | |
| 83/10/04 | 1.9 | .095 | | |
| 83/10/05 | 3.7 | .053 | | |

Note:

(1) Event with less than 4 mm of precipitation.

## 6.0 THE BASE CASE

### 6.1 Seasonal CSO Statistics

The base case analyzed the response of the existing (year 1983) catchments and sewer system in wet weather. The weather conditions were represented by the precipitation data of the selected season, April-October, 1979. The results of the base case provided a direction for formulating CSO control schemes. For example, the results could be used for determining the priority of control of regulators or catchments, and the desirable magnitude of threshold capacities. The results also provided the basis for comparing the effectiveness of the control schemes in CSO reduction. A Summary of results is presented below; detailed results are in Appendix D1.

Estimated CSO frequencies of the regulators are shown in Figure 6.1. Rockcliffe overflowed 26 times, Mt. Dennis and Site 3 each 27 times and Hyde Avenue tank 7 times in the study season. Results of the Berry Road regulator will be discussed later. So the first 3 regulators overflowed in almost every storm event having greater than 4 mm precipitation. Hyde Avenue tank overflowed less frequently. This was expected as the tank was built for reducing overflow. However, since Site 3 and the Hyde Avenue tank both belong to the Hillary catchment, the Hillary catchment in effect overflowed in each of the 27 storm events.

The Hyde Avenue tank overflow outlet and the Site 3, Mt. Dennis and Rockcliffe regulators all discharge to the Black Creek and are located closely together. So, as far as CSO impacts on the Black Creek are concerned, these 3 regulators and the outlet of the tank could be considered as if they were one entity. For convenience, these 4 devices are collectively called the Black Creek group of regulators. One practical significance of the closeness of these devices and the nearly equal CSO frequencies was that, if a control scheme was designed to eliminate overflow in a selected storm event, the scheme should eliminate CSO from the whole group in that event, else the aim of eliminating CSO in that event would be defeated.

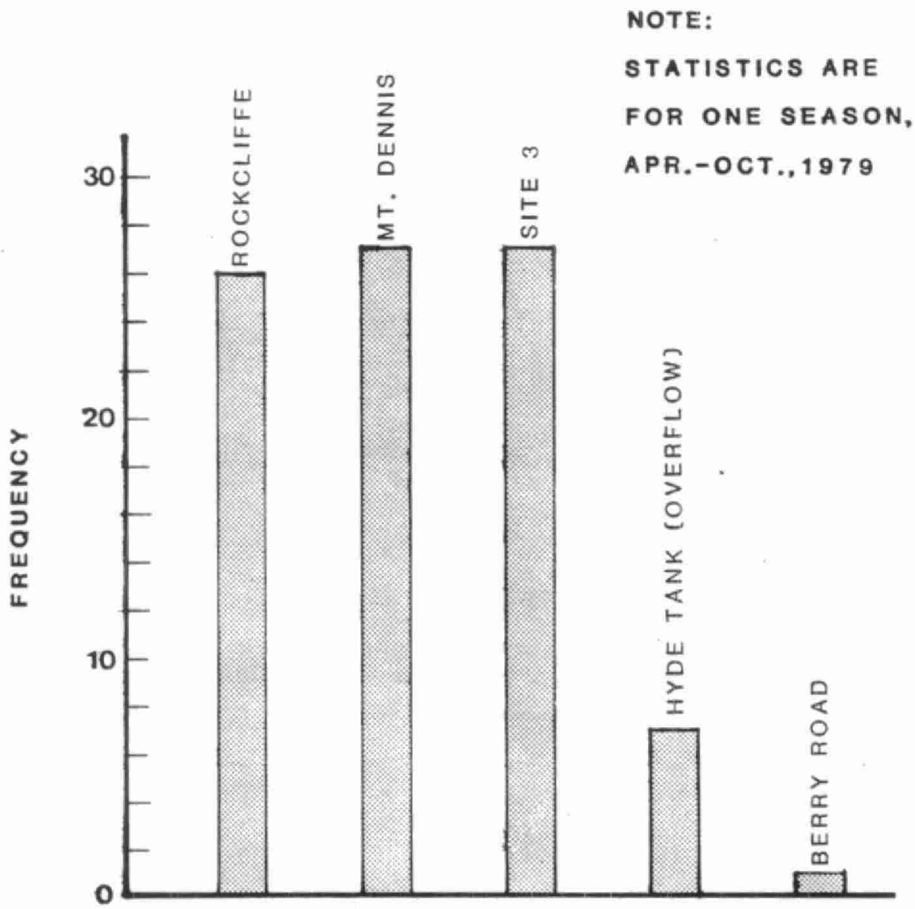



FIGURE 6.1: ESTIMATED OVERFLOW FREQUENCIES
(EXISTING SYSTEM)

The estimated CSO seasonal volume from the Black Creek group of regulators and the seasonal volume stored in the Hyde Avenue tank are shown in the upper diagram of Figure 6.2. Again, the toal overflow volume from the Hillary catchment should be the sum of overflow at Site 3 and the Hyde Avenue tank. With a volume of 280,000 $m^3$, the Hillary catchment was the largest CSO contributor.

The Hillary and the Rockcliffe catchments each discharged about 20% of their combined sewage to the Black Creek in the 27 events and Mt. Dennis discharged 40% of its own. The Hyde Avenue tank stored 8% (102,000 $m^3$) of the combined sewage generated by the Hillary catchment in the 27 events. If this tank were not in existence, the Hillary catchment would have discharged 28% of its combined sewage. The Mt. Dennis discharge of 40% was comparatively high because this catchment has a comparatively higher percentage of surface imperviousness and the regulator's threshold capacity per hectare of catchment is not correspondingly high, as indicated in Table 6.1.

CSO seasonal pollutant loads are summarized in Table 6.2. Seasonal fecal coliform load is omitted because it is a meaningless statistic, but event CSO fecal coliform loads are available (Appendix D1). They ranged from 80,000 billion to 670,000 billion organisms in a single event. The impacts of these pollutants on the Humber river would be studied by another TAWMS project. Time series of CSO flow rates and pollutant concentrations, including fecal coliforms, were provided to that TAWMS project.

The Berry Road regulator is distinct from the Black Creek group of regulators in three ways: it overflowed only in one storm in the season (Figure 6.1); it discharges to the Humber river and not the Black Creek; it is far away (6.1 km) from the nearest regulator (Rockcliffe) of the Black Creek group.

The CSO volume (Table 6.2) from the Berry Road regulator was large, but the storm causing this regulator to overflow was an intense one as will be seen in the next section. The CSO pollutant concentrations at the Berry Road regulator were low. For example,

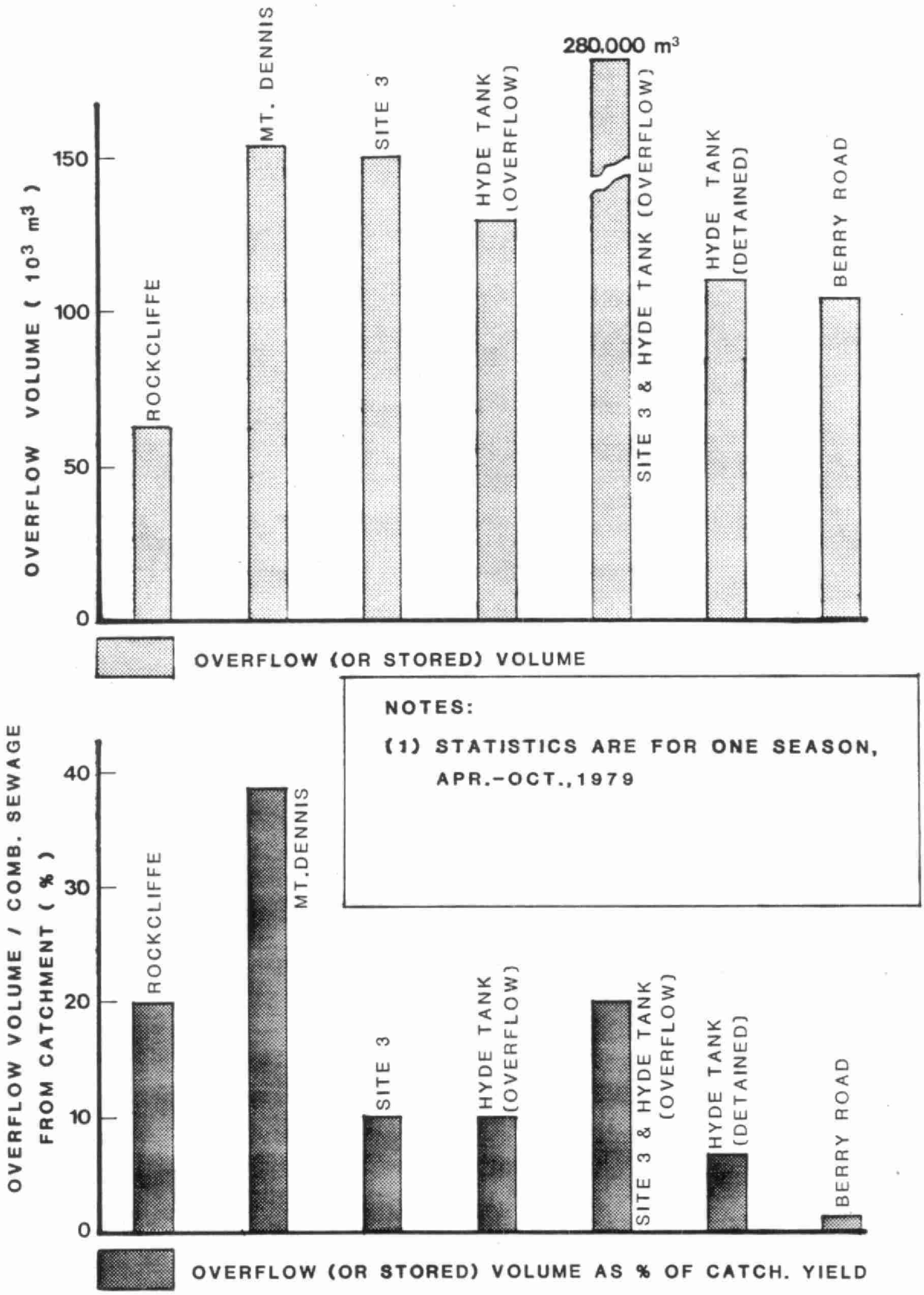



FIGURE 6.2: ESTIMATED OVERFLOW VOLUMES (EXISTING SYSTEM)

TABLE 6.1

COMPARISON OF IMPERVIOUSNESS AND THRESHOLD CAPACITIES

| Catchment | Regulator | Catchment Imperviousness (%) | Threshold Capacity of Regulator (m^3/s/ha) |
|---|---|---|---|
| Hillary | Site 3 | 26.3 | .001724 |
| Mt. Dennis | Mt. Dennis | 49.0 | .001744 |
| Rockcliffe | Rockcliffe | 33.5 | .002397 |

TABLE 6.2

ESTIMATED CSO POLLUTANT LOADS FROM REGULATORS
UNDER EXISTING CONDITIONS (1)

| Overflow | Volume or Load | | |
|---|---|---|---|
| | Black Creek Group Regulators | Berry Road Regulator | Total of All Regulators |
| Volume | 494,000 | 105,000 | 599,000 |
| SS | 97,000 | 4,700 | 101,700 |
| BOD5 | 23,000 | 4,900 | 27,900 |
| Sol-P | 320 | 70 | 390 |
| Tot-P | 1,020 | 140 | 1,160 |
| Cadmium | 4 | 0.4 | 4.4 |
| Copper | 61 | 9 | 70 |
| Lead | 101 | 1.4 | 102.4 |
| Zinc | 176 | 5 | 181 |

Notes:

(1) Catchments and sewers as in 1983.
Season was April - October, 1979.
Volumes in $m^3$; loads in kg.

using results in Table 6.2, the average CSO SS concentration was 45 mg/l (4,700 kg/105,000 $m^3$). Two explanations may be given. First the pollutants on the catchment surfaces were depleted before this intense storm ended. Second, much of the CSO from this regulator was wet-weather I/I from the sanitary sewer area. The I/I was assumed to be free of SS.

There were informal estimates that the Berry Road regulator overflowed more than 20 times in a season and this information was markedly in conflict with the simulation result of one overflow occurrence only. To check the validity of the simulation, two flow monitors were installed in the Humber STS at this regulator in early June 1984 for continuous monitoring until late October 1984. One monitor measured the overflow. The other measured the flow immediately upstream of the regulator. Only one overflow event was observed in the monitored period and it occurred during a thunderstorm (on September 14). The water depth topping the overflow weir crest was less than 10 mm and the overflow duration was about 1 hr. The observation supported the findings of the simulation that the Berry Road regulator would overflow in intense storms only.

## 6.2 Storm Event on July 11, 1979

The intense storm that caused the Berry Road regulator to overflow occurred on July 11, 1979 (Event 790711). It produced large CSO volumes and pollutant loads which weighed heavily in the CSO seasonal statistics as can be seen in Table 6.3. To eliminate CSO in this event would require containment of a CSO volume of 265,000 $m^3$, which is large compared with the volume (7,800 $m^3$) of the existing Hyde Avenue tank or the next largest CSO volume (41,000 $m^3$) in a single event in the season. To control CSO in Event 790711 would require controlling not only the combined sewer area but also the sanitary sewer area, because the CSO in this event arose from both the Black Creek group of regulators (160,000 $m^3$) and the Berry Road regulator (105,000 $m^3$).

TABLE 6.3

ESTIMATED CSO POLLUTANT LOADS IN EVENT 790711
UNDER EXISTING CONDITIONS (1)

| | Volume or Load of All Regulators | | |
|---|---|---|---|
| Overflow | Event 790711 Alone | Seasonal Total (2) | Event 790711 as % of Seasonal Total (2) |
| Volume | 265,000 | 599,000 | 44% |
| SS | 39,000 | 101,000 | 38 |
| BOD5 | 12,000 | 27,900 | 43 |
| Sol-P | 170 | 390 | 43 |
| Tot-P | 460 | 1,160 | 40 |
| Cadmium | 1.7 | 4.4 | 40 |
| Copper | 29 | 70 | 41 |
| Lead | 38 | 102.4 | 37 |
| Zinc | 68 | 181 | 38 |

Notes:

(1) Catchments and sewers as in 1983.
Season was April - October, 1979.
Volumes in $m^3$; loads in kg.

(2) Seasonal total including
Event 790711

In order to arrive at a strategy for handling the CSO in this event, the characteristics of this storm were analyzed. The cause of the large CSO volume produced was that 28.1 mm of precipitation concentrated in one single hour and 35.9 mm in two consecutive hours. The recurrence intervals of these two intensities within the given durations were both 3.3 years as shown in Table 6.4. In contrast, the corresponding recurrence intervals of the storm (June 10, 1979) producing the next largest CSO volume (41,000 $m^3$) were 0.9 and 0.7 year respectively and the recurrence interval of the average intensity was 1.8 years.

In conclusion, the CSO study considered that Event 790711 was an infrequent storm and it was decided that the development of CSO control schemes should exclude consideration of Event 790711. Consequently, all CSO simulation results presented from this point on exclude Event 790711. The base case results presented earlier were adjusted to exclude Event 790711 and the adjusted results are given in Table 6.5. Note that when this event was excluded, the Berry Road regulator did not overflow in the season and that the CSO statistics all pertained to the Black Creek group regulators.

## 6.3 Treatment Loads on Humber WPCP

Averaged over the season, the volume of sewage treated in the Humber WPCP in the base case was 0.526 $m^3$/person/d (including combined sewage) for the combined sewer area and 0.735 $m^3$/person/d for the sanitary sewer area as shown in Table 6.6. It may be inferred that the treatment cost per person was lower for the combined sewer area than the sanitary sewer area, despite the flow of some combined sewage to the WPCP.

In wet weather, the combined sewer area required a higher treatment capacity per person than the sanitary sewer area. The required capacities were .0000298 $m^3$/person/s and .0000164 $m^3$/person/s respectively (1.8:1.0) as shown in Table 6.7.

TABLE 6.4

COMPARISON OF PRECIPITATION DATA OF STORM ON JULY 11, 1979

| | Event on July 11, 1979 | | Event on June 10, 1979 | |
|---|---|---|---|---|
| Attribute | Actual Value | Recurrence Interval (year) | Attribute Value | Recurrence Interval (year) |
| Event precipitation | 36.4 mm | 0.6 | 17.4 mm | <0.3 |
| Event duration | 7 hr | N.C. | 3 hr | N.C. |
| Ante. dry period | 17 hr | N.C. | 61 hr | N.C. |
| Avg. event intensity | 5.2 mm/hr | 1.6 | 5.8 mm/hr | 1.8 |
| 1 hr max. precip. | 28.1 mm | 3.3 | 16.2 mm | 0.9 |
| Max. precip. in 2 consecutive hours | 35.9 mm | 3.3 | 16.4 mm | 0.7 |
| Max. precip. in 4 consecutive hours | 36.1 mm | 2.9 | | |
| Max. percip. in 6 consecutive hours | 36.2 mm | 2.0 | | |

Note: N.C. = Not calculated.

TABLE 6.5

ADJUSTED CSO POLLUTANT LOADS FROM REGULATORS
UNDER EXISTING CONDITIONS (1)

| Overflow | Adjusted Seasonal Total Volume or Load of Black Creek Group Regulators |
|---|---|
| Volume | 334,000 |
| SS | 63,000 |
| BOD5 | 16,000 |
| Sol-P | 230 |
| Tot-P | 690 |
| Cadmium | 2.6 |
| Copper | 41 |
| Lead | 65 |
| Zinc | 112 |

Notes:

(1) Catchments and sewers as in 1983.
Season was April - October, 1979.
Volumes and loads adjusted to exclude Event 790711.
Volumes in $m^3$; loads in kg.

TABLE 6.6

SEWAGE VOLUMES TREATED IN HUMBER WPCP - EXISTING CONDITIONS (1)

| | Estimated Volume to Humber WPCP (m3) | |
|---|---|---|
| | Comb. Sewer Area | San. Sewer Area |
| Storms Greater Than 4 mm | 1,550,000 | 7,340,000 |
| Storms Not Greater Than 4 mm (2) | 320,000 | 0 |
| Volume Returned by Hyde Avenue Tank | 100,000 | 0 |
| Flow in Dry Days | 7,180,000 | 63,220,000 |
| Total in Apr-Oct, 1979 (3) | 9,150,000 | 70,560,000 |
| Volume/Person/Day (4) | 0.526 | 0.735 |

Notes:
(1) Catchments and sewers as in 1983.
Season was April - October, 1979.
(2) Runoff Volume. Assumed to be 0.4 of precipitation.
(3) Excluding event 790711.
(4) Population:
Combined Sewer Area 81,274
Sanitary Sewer Area 448,931

TABLE 6.7

MAXIMUM FLOW RATES TO HUMBER WPCP
UNDER EXISTING CONDITIONS (1)

| | Max. Flow Rate from Sewer Area (m^3/s) | Max. Flow Rate Per Person in Sewer Area (m^3/s/person) (2) |
|---|---|---|
| Combined Sewer Area | 2.43 | .0000298 |
| Sanitary Sewer Area | 7.37 | .0000164 |
| Total of Both Areas | 9.80 | .0000185 |
| WPCP Max. Capacity | 11.8 | .0000223 |

Notes:

(1) Catchments and sewers as in 1983. Maximum flow rate estimated for precipitation data in April-October, 1979. Event 790711 excluded.

(2) Population:
Combined sewer area 81,274.
Sanitary sewer area 448,931.

If the peak primary treatment capacity of the Humber WPCP should, in fact, be taken as 8.9 $m^3/s$ as mentioned in Section 3.4, then the peak primary treatment capacity was exceeded in 7 storm events in the base case. However, the maximum excess in capacity was only 0.9 $m^3/s$ (9.8-8.9) which was 10% of the revised capacity, and the duration of each excess was only a fraction of the storm duration. Therefore, it could be considered that the WPCP had sufficient capacity to treat all the wet weather flow it received.

# 7.0 CSO CONTROL SCHEMES

## 7.1 Introduction

While many methods for CSO control are mentioned in the literature, not many methods have been sufficiently evaluated. A co-author of a recent U.S. Environmental Protection Agency study of case histories of urban stormwater management and technology observed that "measures to control pollution by intermittent stormwater discharges and combined sewer overflow are still in early stages of implementation. Few of these have adequate performance data available." (Finnemore, 1982).

By far the control methods most often used have been structural measures. They include: on-site treatment; off-line and on-line storage; and increase in support in treatment by dry-weather treatment facilities. The above-mentioned U.S. project which studied full scale applications of these measures in Seattle (Washington), Saginaw (Michigan) and Mt. Clemens (Michigan) concluded that the measures, when applied in suitable combinations, were very promising. (Finnemore, 1982).

Most commonly cited examples of on-site treatment include microstraining, air floatation and the swirl concentrator for removal of solids; aerated lagoons for removal of conventional pollutants; chemical coagulation for removal of particulate substances; and disinfection for reduction of bacteria. Although many of these processes are well recognized in the treatment of DWF and show potential for application to treating intermittent flows in pilot-scale studies (Kronis, 1975(?) and 1982; Nebolsine, 1972; Prah, 1979), they are not often used in full-scale application. Possible reasons for their limited use may include high capital cost per kilogram of pollutant removed; uncertainty in results; uncertainty in operational reliability; and the trend of centralizing treatment facilities. Nevertheless, on-site CSO disinfection and screening would be studied in a separate TAWMS project.

"Best management practices" such as street sweeping, catchbasin cleaning and sewer cleaning do not reduce CSO frequency or volume. They are not expected to produce appreciable reduction in pollution loads to the Humber river when applied to the combined sewer area, because the area is only 7% of the Humber river watershed in Metro Toronto. For these reasons, they were not considered.

The following CSO control schemes were developed and considered in this study:

Scheme 1: Detention of overflow at regulators.

Scheme 2: Resetting of regulators, aided/unaided by detention at regulators.

Scheme 3A: Stormwater runoff control (20% catchment reduction) aided/unaided by detention at regulators.

Scheme 3B: Stormwater runoff control (28% catchment reduction) aided/unaided by detention at regulators.

The following 4 alternative methods for reduction of catchment areas were considered in Scheme 3A:

Scheme 3A(i) Detention tanks in local sewers

Scheme 3A(ii) Roof leader disconnection

Scheme 3A(iii) Combined sewer separation

Scheme 3A(iv) Catchbasin inlet restriction

Schemes 1, 2, 3A(i), (ii), (iii) and 3B were found feasible; Scheme 3A(iv) was not. Each of the feasible schemes is self-contained and is an alternative, but not a supplement or a complement, to the others. Each feasible scheme was analyzed to determine the capacities required for various degrees of CSO control, up to complete CSO elimination in the season of April-October, 1979. Note that all schemes considered

Event 790711 as an exception. The maximum recurrence interval of storms in which the CSO control schemes may achieve complete CSO elimination is 1.8 years. All the analyses were carried out with the study model.

In developing the CSO control schemes, engineering judgement was used to see that the schemes were conceptually sound in layout and practical in construction and operation. An order of costs of the schemes was made so that relative merits of the schemes could be evaluated. Unit costs used in the estimates were obtained from the Ministry of the Environment (Braganza, 1985) unless otherwise noted. Costs for engineering services, land and ancillary works such as access road, landscaping of storage sites and instrumentation, were not included in the estimates. A breakdown of the estimates is in Appendix D2. Detailed feasibility study and costing of the schemes would be undertaken by a separate TAWMS project.

The following sub-sections discuss the individual schemes. For convenience of presentation, discussion about the feasibility of integrating measures for basement flooding mitigation into CSO control schemes is reserved until Section 8.0.

## 7.2 CSO Control Scheme 1: Detention of Overflow at Regulators

This scheme provided new detention storage to intercept CSO from the Black Creek regulators. The storage could be made up of one or more tanks as will be discussed later. The new storage would augment the existing Hyde Avenue tank. In general practice, temporary detention of flow has been widely used in treatment of DWF to equalize the flow and hence to reduce the required peak treatment and sewer capacities. This control method applied to combined sewage should be even more beneficial because the peaks of combined sewage are typically many more times higher than the DWF peak. It was assumed that if the new storage was filled, the excess of the intercepted flow would overflow to the Black Creek. It was also assumed that after a storm event, the detained flow would be returned to the Black Creek STS via an underdrain in the storage facility and it would take 24 hours to empty the full storage. This duration was selected to ensure that the

return flow would not overload the sewers or the Humber WPCP by rapid draining and yet the storage facility would be completely emptied before the next storm came. For a total new and existing storage as large as 50,000 $m^3$, the return flow rate would be 0.57 $m^3/s$ which was only 12% of the average design secondary treatment capacity of the Humber WPCP and would not stress the WPCP.

The catchments and sewers simulated in this case were the same as the base case, i.e. conditions existing in 1983.

Four alternative arrangements of the new tank(s), as shown in Figure 7.1, were evaluated. The final choice will depend on required storage capacity, engineering feasibility and costs. Arrangement 1 used one new tank situated downstream of the Rockcliffe regulator. The tank intercepted this regulator as well as the Mt. Dennis and Site 3 regulators. The existing and the new tanks were not interconnected. This arrangement would be suitable if the selected degree of control was low enough such that the storm event to be controlled would not cause the existing tank to overflow. Arrangement 2 was similar to arrangement 1 but the new and existing tanks were interconnected to allow overflow from the existing tank to the new tank. This arrangement and arrangements 3 and 4 could be used for CSO control up to complete CSO elimination. Arrangement 3 used 2 new tanks: one situated as in arrangement 1 and the other near the existing tank to intercept the tank's overflow. The two new tanks were not interconnected. Arrangement 4 used 3 new tanks which were not interconnected. Obviously, this arrangement would be costlier than either arrangement 2 or 3. It was included in case site conditions precluded the choice of the other arrangements.

New sewers would be needed to connect each existing overflow regulator with the new tank(s). The sizes of the connectors are given in Appendix D3. They were designed for the maximum overflow rates in the storms to be controlled. It will not be practical nor justifiable to provide connector sewers of very large sizes to cope with more extreme storm events in other seasons. Therefore, an

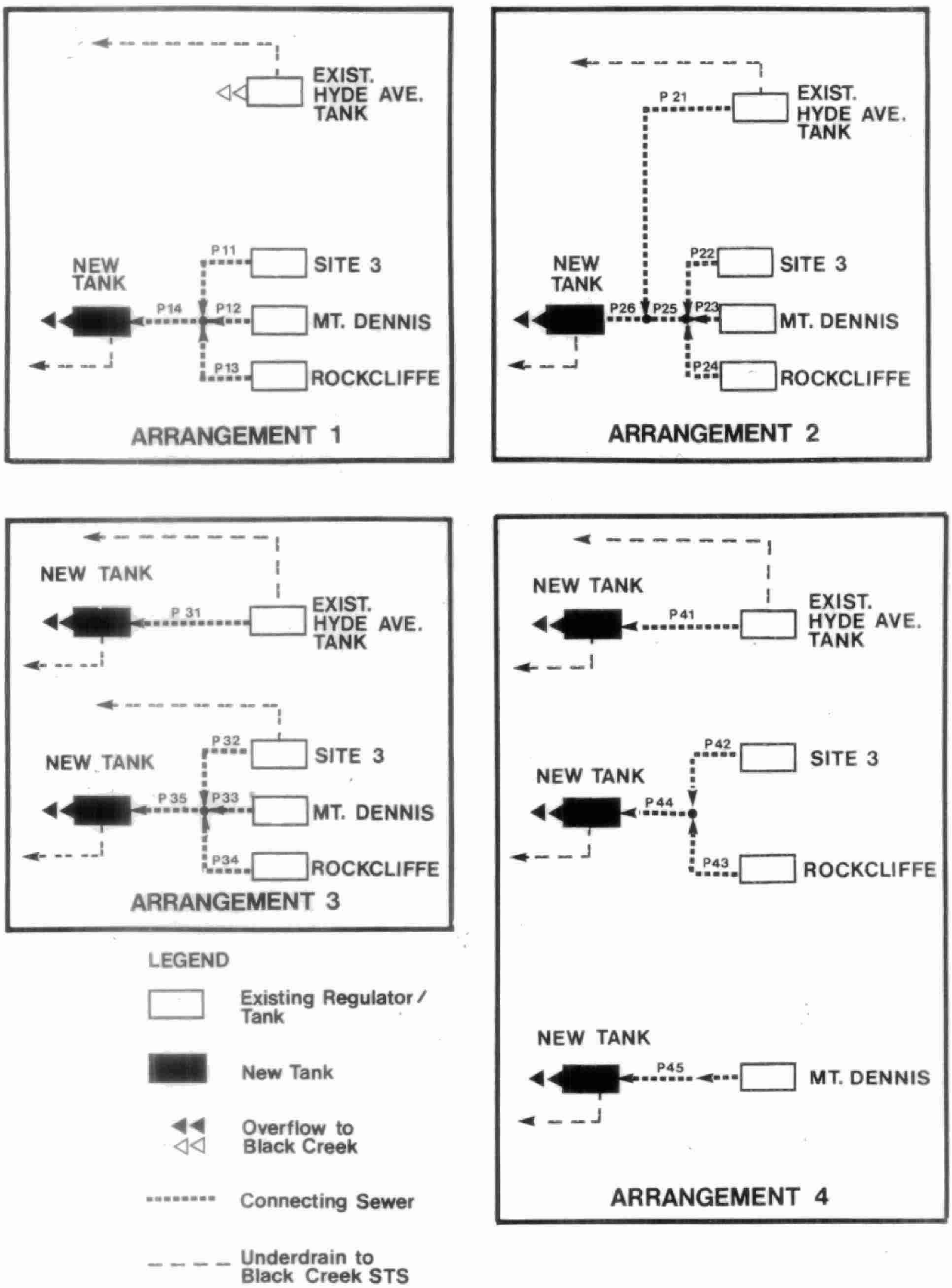



FIGURE 7.1 : ARRANGEMENT OF STORAGE TANKS

emergency overflow opening should still be provided at the upstream end of each connector but the opening should be so designed that overflow will be allowed only when the connector capacity is exceeded.

A series of new storage capacities was examined at increments of 4,000 $m^3$ until CSO was eliminated completely.

### 7.2.1 Performance of Scheme 1

The estimated seasonal CSO frequencies against new storage capacities are shown in Figure 7.2. For required new storage up to 17,000 $m^3$, arrangement 1 would be adequate; the CSO frequency would be reduced from 26 to 7. For required new storage between 17,000 $m^3$ and 37,000 $m^3$, arrangements 2 and 3 were comparable in performance; the CSO frequency would be reduced to between 7 and 3. Complete CSO elimination would require new storage of 41,000 $m^3$ in arrangement 2 and 51,000 $m^3$ in arrangement 3. A larger capacity was required by arrangement 3 because the two unconnected new tanks in this arrangement could not be fully utilized simultaneously. In general, the performance of arrangement 4 was very close to that of arrangement 3. For brevity, results of arrangement 4 are not presented.

It will be noted that, as CSO frequency decreased, a larger increment in capacity was required to reduce the frequency further by 1. Diminution in control efficiency as the degree of control increased, in fact, occurred also in CSO volumes and pollutant loads.

Estimated CSO volumes against new storage are shown in Figure 7.3. Unlike CSO frequency, however, CSO volume decreased with each increment of new storage capacity. This trend of CSO volume reduction would be particularly useful for fecal coliform reduction, because fecal coliform load was proportional to CSO volume. For complete CSO elimination, the seasonal volume of overflow stored and subsequently returned to the Humber WPCP was 334,000 $m^3$. It was less than 4% of the total flow treated by the WPCP during all the 26 events and was about 1/2% of the total volume treated in

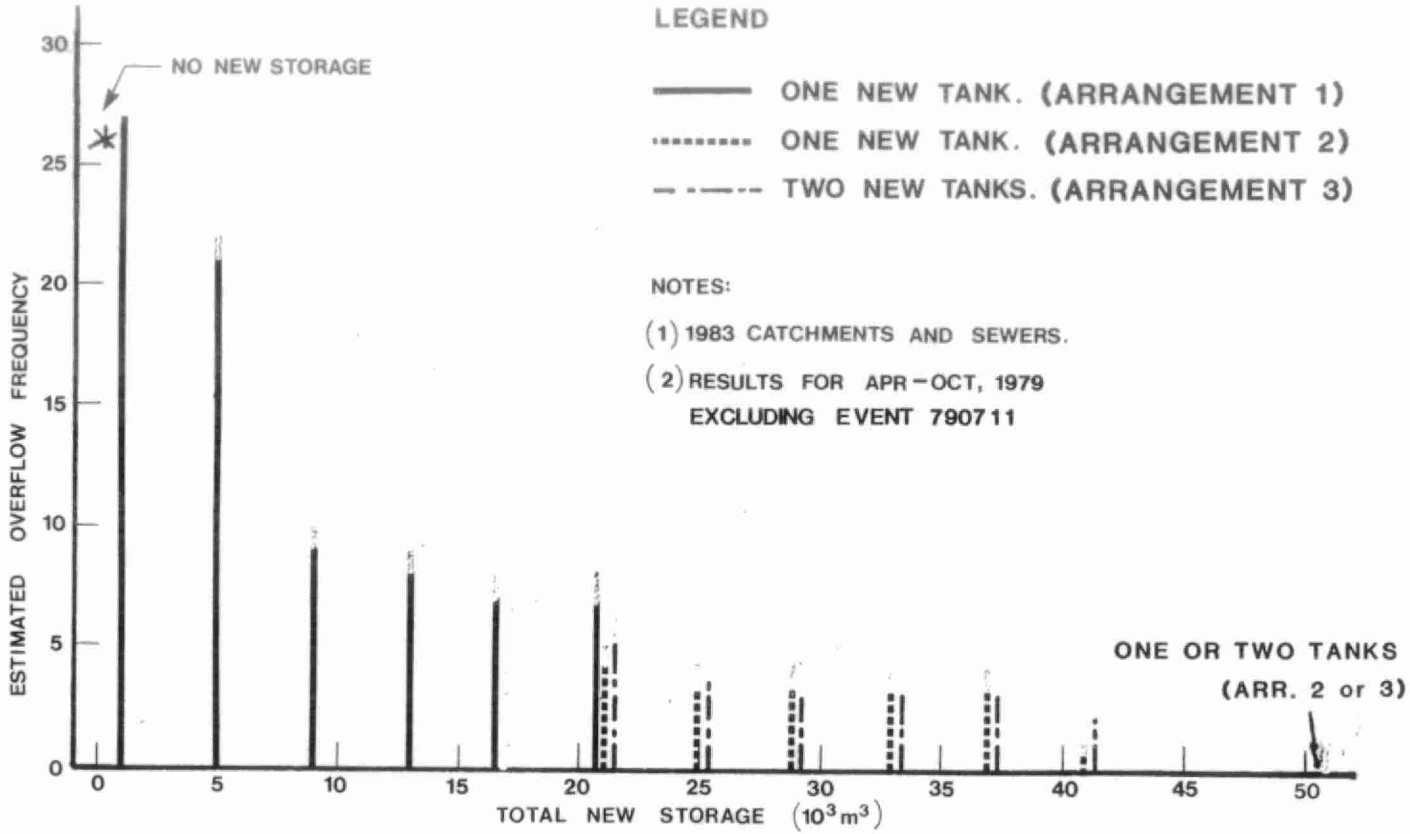
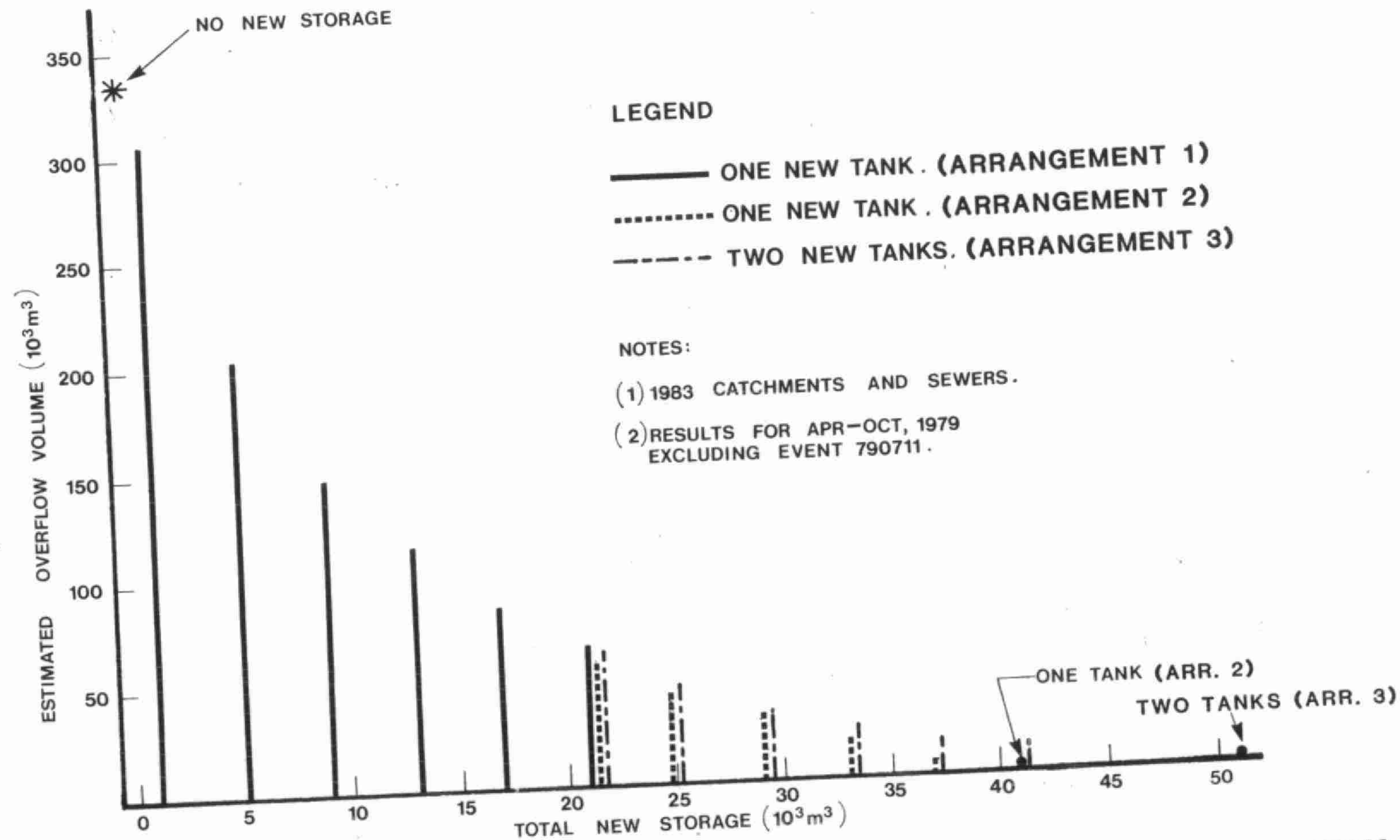



FIGURE 7.2: CONTROL BY STORAGE (BLACK CREEK GROUP)—OVERFLOW FREQUENCY DIAGRAM




STORAGE (BLACK CREEK GROUP)—OVERFLOW VOLUME DIAGRAM

the season. The corresponding increase in treatment cost should be insignificant.

As an illustration of CSO pollutant load reduction, SS reductions are summarized in Table 7.1. The impacts of CSO pollutant load reductions on the qualities of the receiving waters would be studied by a separate TAWMS project.

### 7.2.2 Engineering Considerations and Costs of Scheme 1

From an operational point of view, arrangement 2 is preferable to arrangement 3 because the facilities in arrangement 2 are more centralized. Arrangement 2, however, has 2 apparent disadvantages. First, the connector sewer between the existing tank and the new tank will be laid in a built-up area. Second, the performance of arrangement 2 depends much on the capacity of the long connector sewer (1.5 km in length) between the existing and new tanks. In a season other than the study season, the precipitation intensities of some storms may exceed the intensities of the storms in the study season even though the precipitation depths of the storms in the two seasons are comparable. Precipitation intensities often vary much more widely than precipitation depths. When the prevailing precipitation intensity exceeds the design intensity, it will be difficult for the connector sewer to accommodate the excessive flow peaks, because the frictional resistance to flow in a long pipeline is proportionally large. The consequence would be that CSO could occur at the emergency opening more often than expected.

The order of costs for complete elimination of CSO is shown below:

| Tank Arrangement 2 | Order of Cost |
|---|---|
| Enclosed tank, 41,000 $m^3$ near Site 3 | $2.2 million |
| Connector sewer, 2 m dia x 1.5 km | 1.7 |
| Total | 3.9 |
| **Tank Arrangement 3** | |
| Enclosed tank, 16,000 $m^3$ near Hyde Avenue | $0.9 million |
| Enclosed tank, 35,000 $m^3$ near Site 3 | 1.9 |
| Total | 2.8 |

Arrangement 3 is therefore more preferable.

TABLE 7.1

ESTIMATED SS OVERFLOW LOADS
CONTROL BY STORAGE (1)

| New Storage Capacity (m3) | SS Overflow Load (kg) | SS Overflow Load as % of No Control |
|---|---|---|
| 0 | 63,000 | 100 % |
| 1,000 | 47,000 | 75 |
| 5,000 | 38,000 | 60 |
| 9,000 | 31,000 | 49 |
| 13,000 | 24,000 | 38 |
| 17,000 | 18,000 | 29 |
| 21,000 | 13,000 | 21 |
| 25,000 | 10,000 | 16 |
| 37,000 | 1,770 | 3 |
| 41,000 | 0 | 0 |

Note :

(1) Catchments and sewers as in 1983.
Totals for April - October 1979,
excluding 790711

It is recommended that future feasibility study and design of any CSO storage tank (in this scheme or the other schemes) should consider the matters mentioned below:

An enclosed tank should be force-ventilated.

The inlets and emergency overflow device of a tank should be designed to ensure that the influent sewers will not be surcharged when the tank is filled to its high-water level and that the overflow device will not be flooded by the Black Creek at high flow. Preferably, the tank should be able to return the detained flow to the Black Creek STS by gravity. If this is not feasible, return flow pumping should be provided.

Since the capacity of the tank will be exceeded in a storm that is more intense than the design storm, consideration should be given to the need of disinfection of the overflow to the Black Creek.

An access road for heavy maintenance machinery to reach the tank is essential. Adequate facilities for maintenance and operation of the tank should be provided at the tank site. Flow monitoring instrumentation at the tank site should be provided for monitoring performance of the tank.

## 7.3 CSO Control Scheme 2: Resetting of Regulators

This scheme increased combined sewage flow to the Humber WPCP during wet weather by resetting the regulators. The concurrent use of new storage to intercept overflow at the regulators was also examined. The new storage tanks were assumed to be arranged in the same manner as in the previous scheme.

Scheme 2 would not be feasible if the peak primary treatment capacity of the WPCP should be revised from 11.8 $m^3/s$ to 8.9 $m^3/s$ as mentioned in Section 3.4. Scheme 2 presented in this section assumed the WPCP to have the original primary treatment capacity of 11.8 $m^3/s$.

The maximum allowable total increase in combined sewage flow to the WPCP was set at 2.0 $m^3/s$ which is about 17% of the Humber WPCP peak capacity. The amount of increased flow was determined subject to 2 constraints: that the peak primary treatment capacity of the WPCP should not exceed 11.8 $m^3/s$ and that sewage levels in the sewers upstream of the regulators would not be raised. Primary effluent in excess of the secondary treatment capacity would bypass the secondary treatment process to the effluent outfall after chlorination.

The threshold capacities of the regulators, before and after resetting to accommodate the increased flow, are indicated below:

| | Before Resetting | After Resetting | Increase |
|---|---|---|---|
| Site 3 | 1.64 $m^3/s$ | 2.64 $m^3/s$ | 1.00 $m^3/s$ |
| Mt. Dennis | 0.32 | 1.12 | 0.80 |
| Rockcliffe | 0.47 | 0.67 | 0.20 |
| Berry Road | 10.20 | No change | 0 |

A proportionally higher increase was given to the Mt. Dennis regulator because this regulator discharged a larger percentage of its combined sewage to the Black Creek than did the other regulators.

The capacity of the existing Black Creek STS between the Humber STS and Site 3 is just sufficient for the maximum peak flow of 4.3 $m^3/s$ of the base case. Scheme 2 assumed a duplication of this section of the Black Creek STS to provide the additional capacity of 2.0 $m^3/s$ to prevent surcharge of the existing sewers upstream of the regulators.

### 7.3.1 Performance of Scheme 2

If resetting of regulators was used alone, CSO frequency would be reduced from 26 to 19 (Figure 7.4) and CSO volume from 334,000 $m^3$ to 92,000 $m^3$ (Figure 7.5) in a season. The reason for the marked difference in CSO frequency and volume performances was that resetting of regulators was able to reduce the overflow volumes

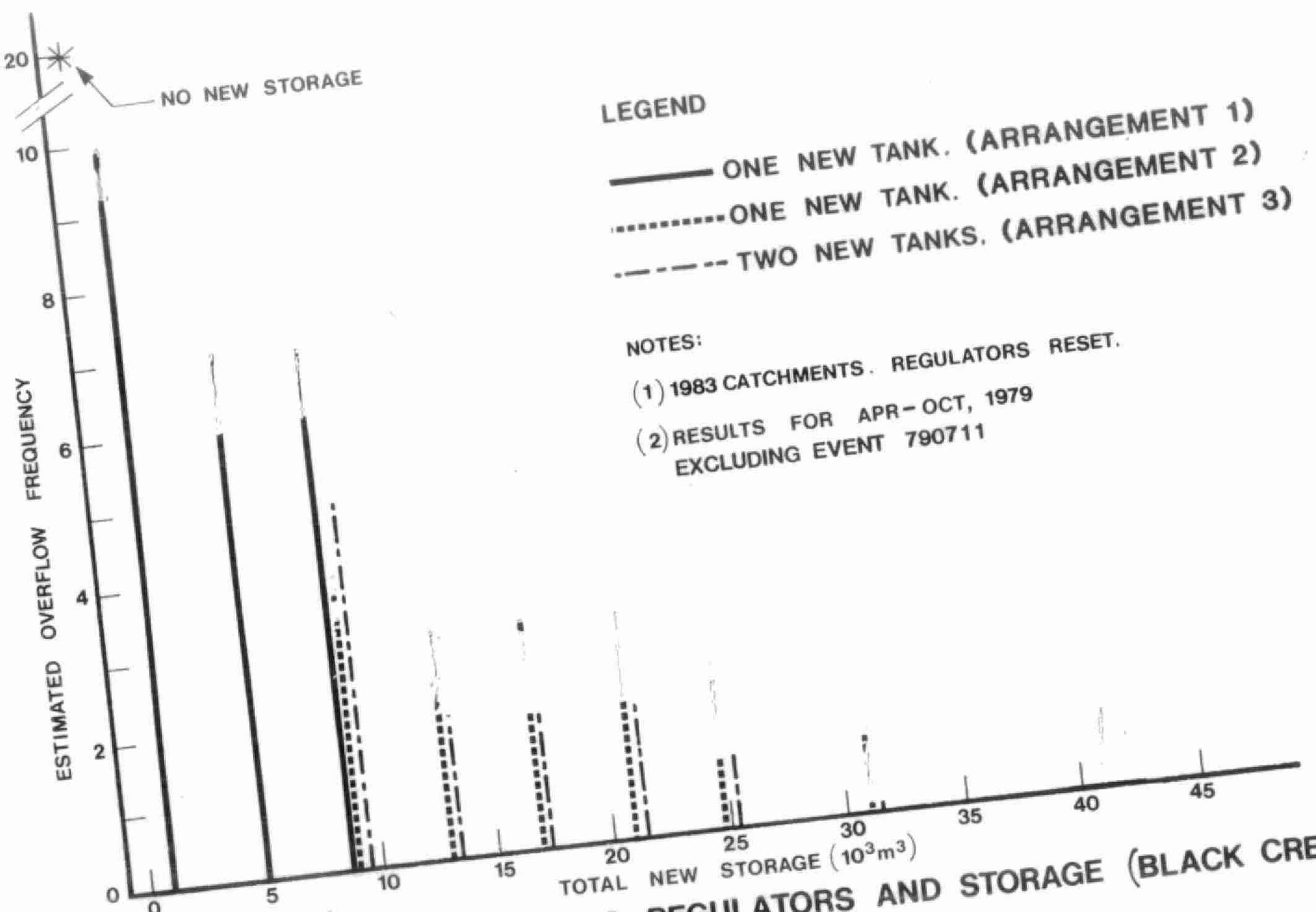



FIGURE 7.4: CONTROL BY RESETTING REGULATORS AND STORAGE (BLACK CREEK GROUP) —OVERFLOW FREQUENCY DIAGRAM

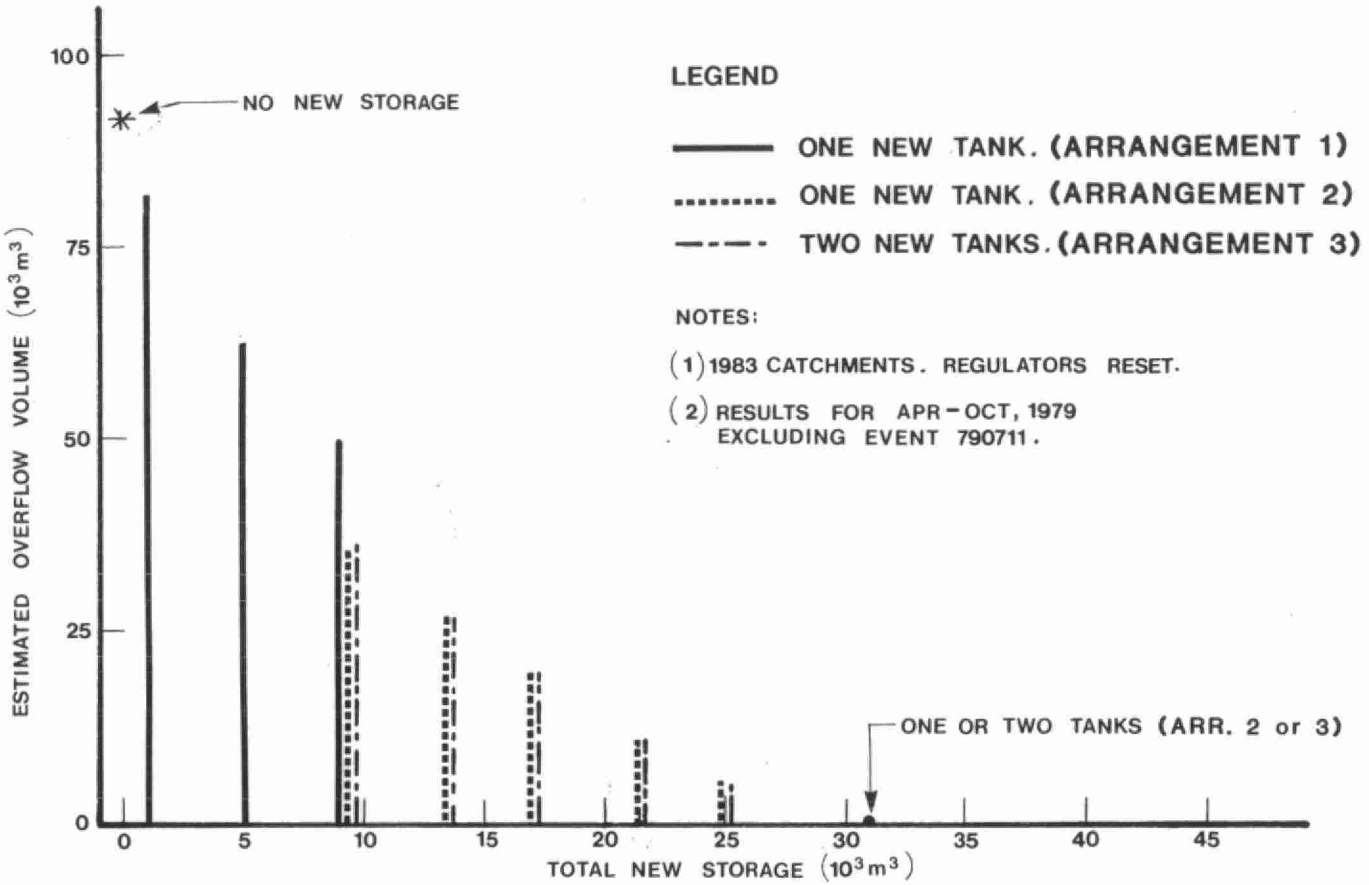



**FIGURE 7.5: CONTROL BY RESETTING REGULATORS AND STORAGE (BLACK CREEK GROUP) —OVERFLOW VOLUME DIAGRAM**

substantially but the reduction was not enough to completely eliminate a number of CSO occurrences. For CSO control beyond the degrees indicated above, other methods would be required to augment the resetting of regulators. New storage was used for this purpose.

Compared with Scheme 1, Scheme 2 required a lesser amount of new storage to achieve the same degree of control. This was expected. For example, to eliminate CSO completely, Scheme 2 would require only 31,000 $m^3$ new storage compared with 51,000 $m^3$ in Scheme 1. For complete CSO elimination using Scheme 2, the seasonal increase in combined sewage volume treated by the WPCP would be 334,000 $m^3$, the same as Scheme 1. In Scheme 2, however, 242,000 $m^3$ of this volume would receive primary treatment only, since it was brought to the WPCP during storms as the result of resetting the regulators. The remaining 92,000 $m^3$ would be sewage returned from storage to the WPCP after storms and would receive full treatment.

As an illustration of CSO pollutant load reductions, SS load reductions are summarized in Table 7.2. The resetting of regulators reduced the CSO SS load from 63,000 kg to 23,000 kg, a reduction of 63%.

### 7.3.2 The Berry Road Regulator

In Scheme 2, the Berry Road regulator overflowed for one hour in each of two events other than Event 790711. The hydrographs of the worse of these 2 events are shown in Figure 7.6. The overflow rate was only about 6% of the prevailing flow rate in the sewer. The CSO volume was about 4,000 $m^3$ in each event. Note that the overflow was less than the expected error in hydrologic modelling, and should be considered as an uncertain result.

It is suggested that provision of a detention tank to intercept the overflow of 4,000 $m^3$ should be withheld unless further flow monitoring confirms the need of this tank. If it is required, two alternative locations may be considered at the Berry Road regulator; or in the WPCP compound at the plant inlet. The choice will depend on detailed feasibility study.

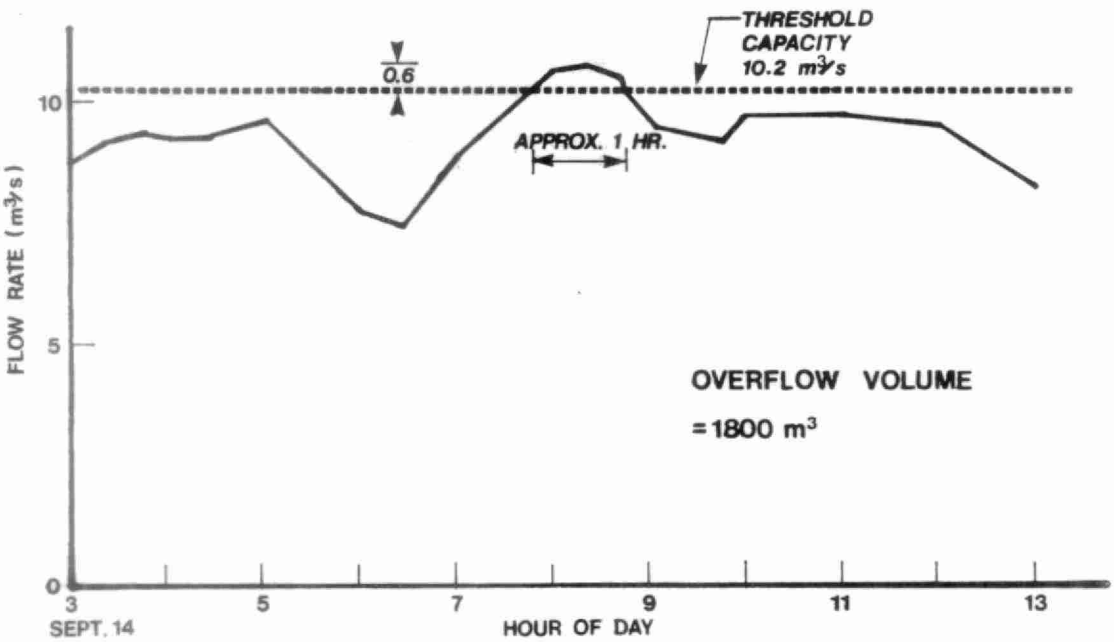
TABLE 7.2

ESTIMATED SS OVERFLOW LOADS
RESETTING REGULATORS (1)

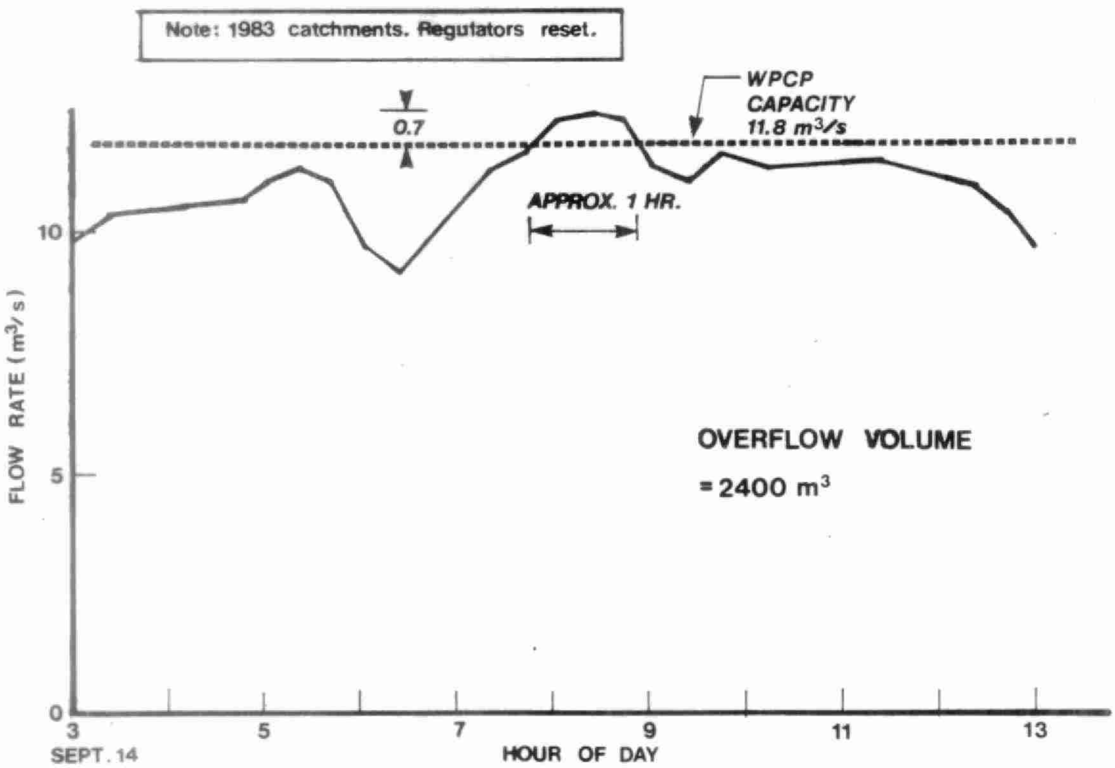
| In Combination With New Storage (m3) | SS Overflow Load (kg) | SS Overflow Load as % of No Control |
|---|---|---|
| 0 | 23,000 | 37 |
| 1,000 | 20,000 | 32 |
| 5,000 | 15,000 | 24 |
| 9,000 | 10,000 | 16 |
| 13,000 | 8,000 | 13 |
| 17,000 | 5,650 | 9 |
| 21,000 | 3,470 | 6 |
| 25,000 | 1,800 | 3 |
| 37,000 | 0 | 0 |
| 41,000 | 0 | 0 |

Note :

(1) Catchments as in 1983. Regulators reset.
Totals for April - October 1979, excluding 790711.




(A) *AT UPSTREAM END OF BERRY RD. REGULATOR*




(B) *AT UPSTREAM END OF WPCP INLET*

FIGURE 7.6: HYDROGRAPHS OF EVENT 790913-14

### 7.3.3 Feasibility Considerations and Costs of Scheme 2

The order of costs of Scheme 2 for complete elimination of CSO is shown below:

| | Order of Cost |
|---|---|
| Enclosed tank, 16,000 $m^3$ near Hyde Avenue | 0.9 million |
| Enclosed tank, 15,000 $m^3$ near Site 3 | 0.9 |
| Enclosed tank, 4,000 $m^3$, at Berry Road | 0.4 |
| Black Creek STS duplication, 1.2 m dia x 2.1 km | 1.0 |
| Special allowance | 0.3 |
| Total | 3.5 |

The special allowance was made for two factors. About 500 m of the duplication sewer would lie across a golf course. Extra costs would probably be incurred for temporary reprovisioning work during sewer construction and for final rehabilitation of the disturbed golf course. As well, the duplication sewer would run along the Black Creek where soil conditions and ground water table could increase construction costs.

Scheme 2 would cost more than Scheme 1. In wet weather, Scheme 2 would stress the WPCP capacities to the limit. There would be no spare capacity to cope with any contingency or a possible future increase in flow due to urban redevelopment at a higher density than existing densities or due to an increase in water consumption.

In summary, Scheme 2 is less competitive than Scheme 1.

## 7.4 CSO Control Scheme 3: Stormwater Runoff Control

This scheme reduced CSO by controlling stormwater runoff in the combined sewer catchments or in the local combined sewers. A number of methods for controlling stormwater runoff are mentioned in the literature. The more promising ones, which were considered in Scheme 3, are listed below:

1. Detention of flow in local combined sewers;

2. Disconnecting roof leaders from combined sewers and infiltrating stormwater from the roofs into pervious surfaces around houses;

3. Combined sewer separation; and

4. Inlet restriction of stormwater at catchbasins.

Each of these methods is equivalent to reducing the size of the combined sewer catchment.

Reduction of combined sewer area was examined for 2 cases: 20% in Scheme 3A and 28% in Scheme 3B. The percentages of catchment reductions were selected arbitrarily, but the selection had been planned to obtain CSO control within a practical range of runoff control. If other degrees of CSO control are wanted, results may be obtained by interpolation of the available simulation results or by additional simulations.

The analysis approach was that the study model was first used to estimate CSO reductions resulted from the catchment reductions. Then the extent of each of the above 4 methods required for achieving the catchment reductions was estimated. Finally, an order of costs of control using these methods was determined and the relative merits of the methods were evaluated.

A breakdown of catchment reductions is shown in Table 7.3. The "Remainder" and the Kitchener sub-catchments of the Hillary catchment (Figure 3.1) were selected for control because basement flooding complaints arose most frequently in these sub-catchments. Larger percentages of catchment reduction were applied to the Mt. Dennis and Rockcliffe catchments to take advantage of their higher percentages of imperviousness.

In both Schemes 3A and 3B, the use of stormwater runoff control alone and in combination with new storage at regulators was studied.

TABLE 7.3

CATCHMENT REDUCTIONS IN CONTROL SCHEMES 3A AND 3B

| Catchment | Area Before Reduction (ha) | Reduction in Scheme 3A | | | Reductions in Scheme 3B | | |
|---|---|---|---|---|---|---|---|
| | | Gross Area (%) | Gross Area (ha) | Impervious Area (ha) | Gross Area (%) | Gross Area (ha) | Impervious Area (ha) |
| Kitchener | 230.4 | 0 | 0 | 0 | 20 | 46.1 | 12.1 |
| "Remainder" | 514.7 | 20 | 102.9 | 27.1 | 30 | 154.4 | 40.6 |
| Keele | 137.8 | 0 | 0 | 0 | 0 | 0 | 0 |
| Mt. Dennis | 167.0 | 40 | 66.8 | 32.7 | 40 | 66.8 | 32.7 |
| Rockcliffe | 196.1 | 40 | 78.4 | 26.3 | 40 | 78.4 | 26.3 |
| Total | 1,246.0 | 20 | 248.1 | 86.1 | 28 | 345.7 | 111.7 |

### 7.4.1 Performances of Schemes 3A and 3B

In Scheme 3A, if runoff control was used alone, i.e. without new storage at regulators, CSO frequency was not reduced at all (Figure 7.7) but CSO volume was reduced by 50% to 165,000 $m^3$ (Figure 7.8). The marked difference between CSO frequency and CSO volume performances indicated that although volume reduction was substantial, the reduction was not sufficient to eliminate CSO occurrence in any of the storm events. As mentioned earlier, volume reduction should be particularly useful for the reduction of CSO fecal coliform loads. To obtain CSO control beyond the results indicated above, new storage was used to intercept overflow from the regulators. The results were very similar to those of Scheme 2, as can be seen by comparing Figure 7.4 with Figure 7.7, and comparing Figure 7.5 with Figure 7.8. Complete CSO elimination would require new storage of 29,000 $m^3$ at the regulators.

As an illustration of reduction in CSO pollutant loads by Scheme 3A, SS loads are shown in Table 7.4. Again, the results were very similar to Scheme 2's.

Results of Scheme 3B are shown in Figures 7.9 and 7.10 and Table 7.4. Despite the larger reduction in catchment areas, the CSO reduction performances of Scheme 3B were not much better than the performances of Scheme 3A. Therefore, it may be expected that the effectiveness of runoff control in reducing CSO would diminish rapidly beyond the extent of catchment reduction in Scheme 3B.

In view of the above findings, only Scheme 3A was considered further for the application of runoff control methods to produce the catchment reduction effect. The results are discussed in subsections 7.4.2 through 7.4.5.

### 7.4.2 Scheme 3A(i): Detention of Flow in Local Combined Sewers

This runoff control method assumed that underground tanks were installed in local combined sewers to detain the stormwater runoff from the catchment area deleted in model simulation. The detained

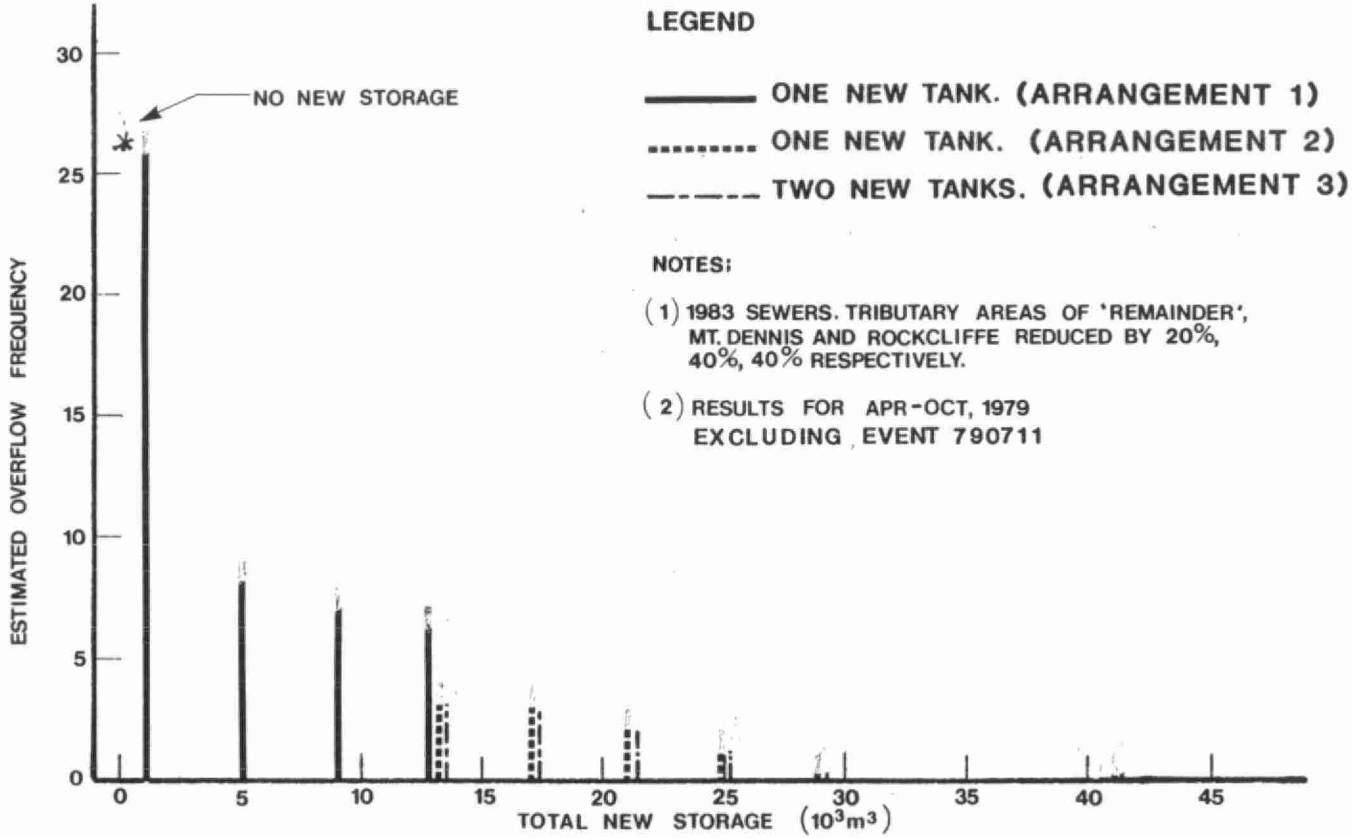



FIGURE 7.7: RUNOFF CONTROL AND STORAGE (A) — OVERFLOW FREQUENCY DIAGRAM

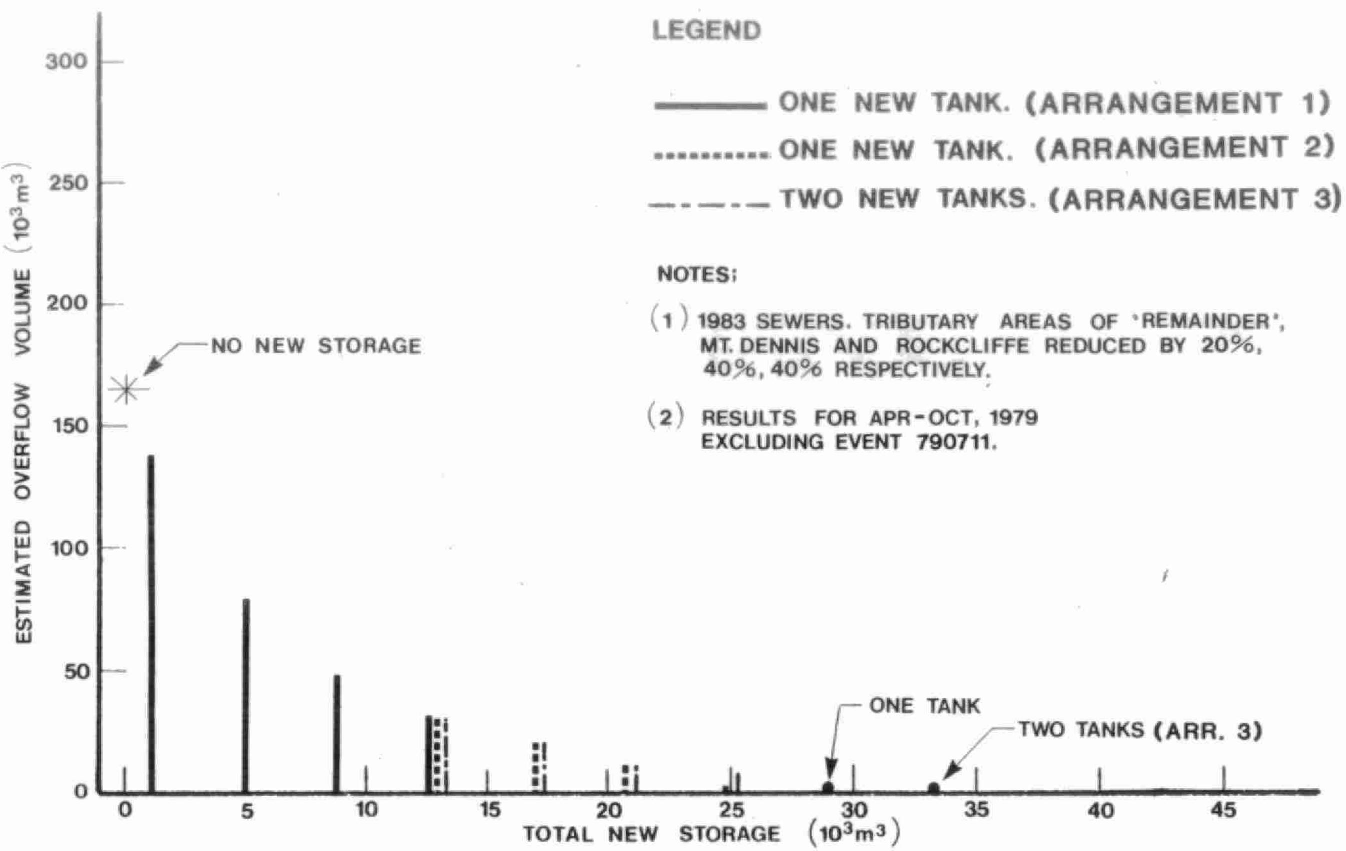



FIGURE 7.8 : RUNOFF CONTROL AND STORAGE ( A ) — OVERFLOW VOLUME DIAGRAM

TABLE 7.4

ESTIMATED SS OVERFLOW LOADS
RUNOFF CONTROL (1)

| In Combination With New Storage (m3) | Scheme 3A | | Scheme 3B | |
|---|---|---|---|---|
| | SS Overflow Load (kg) | SS Overflow Load as % of No Control | SS Overflow Load (kg) | SS Overflow Load as % of No Control |
| 0 | 26,000 | 41 | 20,000 | 32 % |
| 1,000 | 18,000 | 29 | 13,000 | 21 |
| 5,000 | 13,000 | 21 | 8,000 | 13 |
| 9,000 | 8,000 | 13 | 5,000 | 8 |
| 13,000 | 5,000 | 8 | 3,000 | 5 |
| 17,000 | 3,000 | 5 | 2,000 | 3 |
| 21,000 | 2,000 | 3 | 400 | 1 |
| 25,000 | 400 | 1 | 0 | 0 |
| 37,000 | 0 | 0 | 0 | 0 |

Note :

(1) Catchments assumed reduced. Regulators as in 1983.
Totals for April - October 1979, excluding 790711.

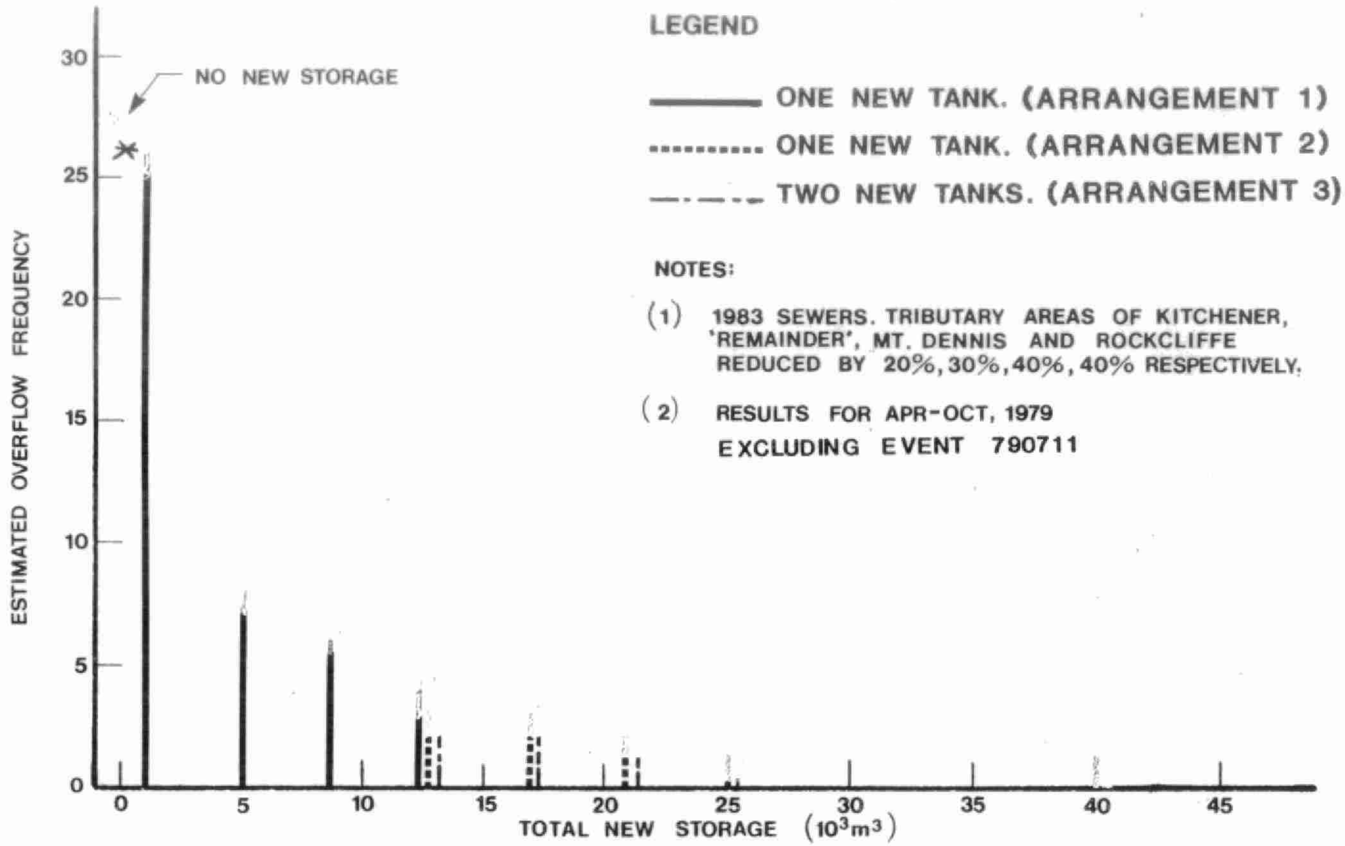



FIGURE 7.9 : RUNOFF CONTROL AND STORAGE (B) — OVERFLOW FREQUENCY DIAGRAM

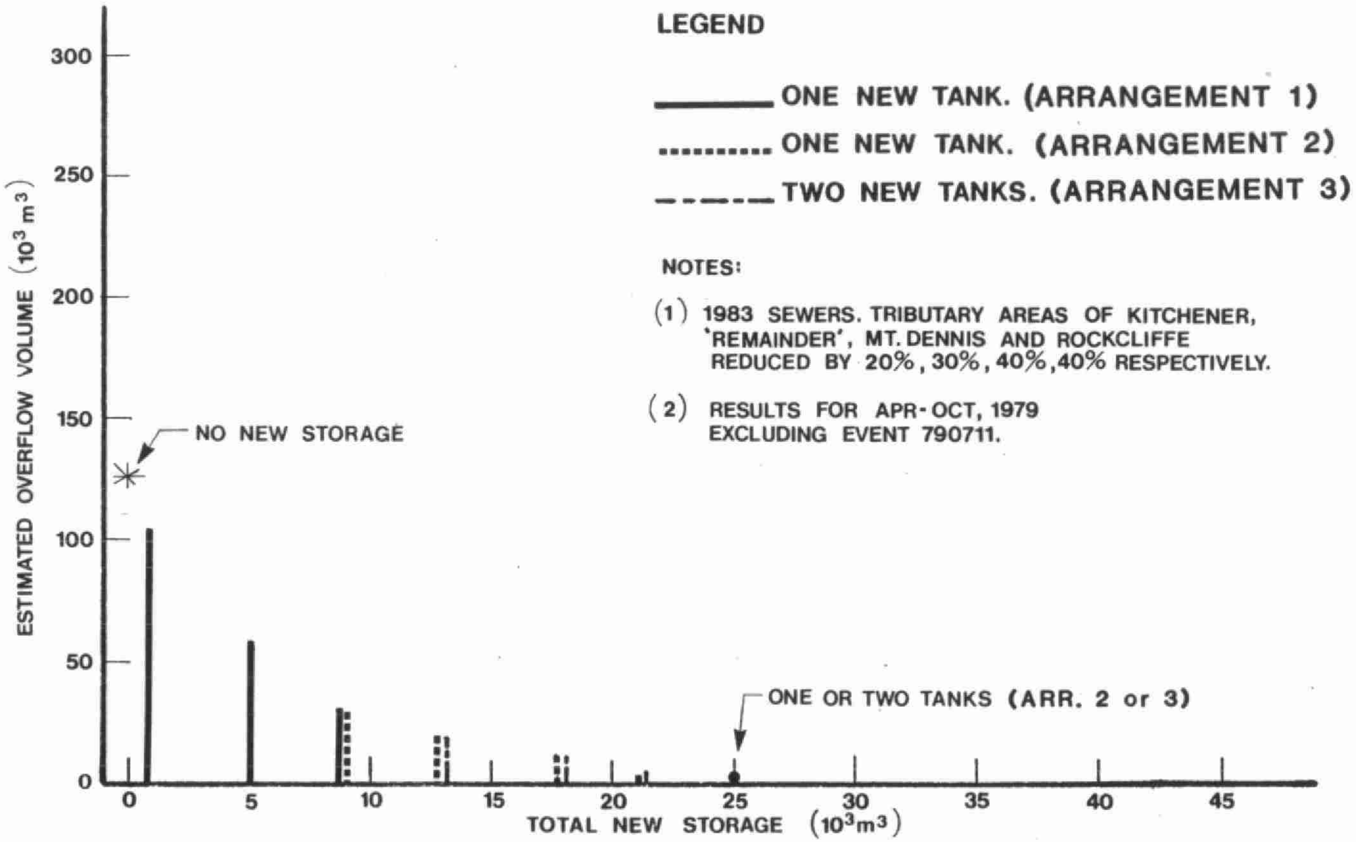



FIGURE 7.10: RUNOFF CONTROL AND STORAGE (B) — OVERFLOW VOLUME DIAGRAM

runoff was assumed to be returned to the local combined sewers after the storm.

This method is a development in recent years and is gaining acceptance. So far, this method has been used for local flood control only and the City of York has used it in a limited scale for the same purpose. Typically, a local detention tank is built underground adjoining the sewer to be controlled and is made of a short length of a large-diameter pipe. A manhole provides access to the tank. Flow control valves are installed inside the manhole.

The total capacity of local detention tanks required was 14,882 $m^3$. It was estimated as the difference between the total flow volume (WPCP treated volume plus overflow volume) of Scheme 3A and the volume of the base case in the most critical event. Assuming each local detention tank to be a pipe of 2.5 m in diameter and 60 m in length, its capacity would be 295 $m^3$ and 50 tanks would be required. The order of cost for complete CSO elimination would be:

| | Order of Cost |
|---|---|
| 50 local tanks at $90,000 | 4.5 million |
| 29,000 $m^3$ new storage near Site 3 | 1.6 |
| Total | 6.1 |

This estimate is more than twice Scheme 1's cost of $2.8 million. Except for high cost and the need for routine cleaning and maintenance of the tanks, this scheme does not have apparent serious disadvantages.

### 7.4.3 Scheme 3A(ii): Roof Leader Disconnection

It is well known that an increase in surface imperviousness of a catchment, such as due to urbanization, will increase stormwater runoff. It is often postulated that the converse of this phenomenon should be applied to reduce stormwater runoff in a developed catchment. One classic application, which was considered for Scheme 3A(ii), is disconnection of roof leaders of houses and

infiltrating the stormwater from roofs into pervious surfaces around the houses.

The area of impervious surfaces to be reduced was 86.1 ha. To obtain this reduction would require disconnection of roof leaders of 5,740 houses, assuming the average roof area to be 150 $m^2$ per house. The order of costs for complete CSO elimination would be:

| | Order of Cost |
|---|---|
| Disconnecting 5,740 houses at $256 (Crozier, 1984) | $1.5 million |
| 29,000 $m^3$ new storage near Site 3 | 1.6 |
| Total | 3.1 |

This estimate is conservative because the actual average roof area is probably smaller than 150 $m^2$ per house. In any case, Scheme 3A (ii) would cost more than Scheme 1 ($2.8 million).

This scheme is marginally feasible. The reliability of roof leader disconnection in this large scale is uncertain. Literature reporting on disconnection of existing roof leaders is scarce and reported cases seem to be concerned with basement flooding mitigation and not CSO control (Vatagoda, 1982). A 100% runoff reduction from the disconnected roofs cannot be expected for a number of reasons. For example, not all the receiving pervious surfaces are flat; some stormwater may run off from the sloping surfaces to sewers before the stormwater can infiltrate into the ground. Another possible reason is that the discharge from a roof leader may not spread on sufficient pervious surface for all the stormwater to infiltrate into the ground. In a roof leader disconnection demonstration project in the City of Stratford, the experimentor reported that no clear indication of flow reduction was observed (Crozier, 1984).

A potential problem of roof leader disconnection is seepage of surface water into basements due to poor surface drainage and flow blockage caused by snow.

Public acceptance of the scheme is uncertain and should be ascertained. The existing sewer bylaws of the City of York do not have specific provisions governing the disposal of stormwater from roofs, except for buildings erected after 1978 and buildings in a few designated streets where houses are prone to flooding.

In summary, this scheme should be considered for adoption only after the reliability and effectiveness of roof leader disconnection have been proven beyond doubt in a large scale pilot project.

### 7.4.4 Scheme 3A(iii): Combined Sewer Separation

The City of York is implementing a long-term combined sewer separation program developed in the late 1960's (Gore and Storrie Ltd., 1968). The program provides new local storm sewers at shallow depths (above existing sewers) to convey without surcharge 70% of the tributary storm drainage, and new trunk sewers at greater depths (below existing sewers) to convey 100% of the tributary storm drainage. The new trunk sewers are designed for a 2-year storm frequency using a City of Toronto (1965) standard. The storm is equivalent to a rainfall of intensity of 91.4 mm/hr for a duration of 8 minutes (Gore and Storrie Ltd, 1968).

Scheme 3A(iii) would require sewer separation in 248.1 ha (Table 7.3). The average cost of sewer separation in 1968 was $16,800/ha (Gore and Storrie Ltd, 1968). This cost updated to 1984 dollar value by a Canadian construction index of 297% (Fortin, 1985), would be $49,900/ha. Then the cost of the Scheme would be:

| | Order of Cost |
|---|---|
| Sewer separation in 248.1 ha at $49,900 | 12.4 million |
| 29,000 $m^3$ new storage near Site 3 | 1.6 |
| Total | 14.0 |

The above estimate is conservative because Scheme 3A(iii) assumed 100% separation but the original unit cost was based on 70% separation. In any case, the estimate already demonstrates sufficiently that the cost of sewer separation would be several times higher than the cost of Scheme 1.

Apart from high cost, sewer separation has 3 major disadvantages:

1. The sewer construction work will disrupt existing neighbourhoods extensively.

2. A sewer separation program typically takes many years to complete. Pending the final completion, new sewers laid during interim periods may not be able to function fully.

3. The runoff pollutants will still be transported by the new storm sewers to receiving waters. If the combined sewers are not separated and the CSO is intercepted, the pollutants will be treated at the WPCP. The estimated seasonal SS load discharged by the new storm sewers to the Black Creek in Scheme 3A(iii) was 67,000 kg, slightly more than the 63,000 kg CSO SS load before the sewer separation.

Sewer separation has one advantage in that the stormwater discharged from the storm sewers would contain much fewer fecal coliform bacteria than would CSO and the stormwater normally would not be contaminated with sanitary sewage. It should be noted, however, that this advantage will no longer exist if the CSO is eliminated by other schemes.

### 7.4.5 Scheme 3A(iv): Inlet Restriction of Stormwater at Catchbasins

This method controls the rate of stormwater runoff entering catchbasins from roads. The restriction is achieved by increasing the spacing of catchbasins in the case of a new development or by sealing some catchbasins in the case of an existing development. Restriction may be further augmented by installing an orifice in a catchbasin so that stormwater runoff entering a sewer via the catchbasin cannot exceed a pre-determined rate. The rejected stormwater runoff will use the road as its conduit. Consequently, this control method requires an outlet to be made available at the low spot of a road so that the runoff flowing down the road can

drain to a storm sewer, a watercourse or a detention facility, for example, a temporary pond in a park.

This method is a companion development of the local detention tank method. It has been used in some local sewer systems.

This method was not considered suitable for the combined sewer area of the present study because the road layout was not designed to integrate this use and the topography is generally unfavourable to the method. Therefore, this method was not evaluated further.

### 7.4.6 Conclusion of Runoff Control Methods

Except for the catchbasin inlet restriction method, all the runoff control methods are likely to be feasible for CSO control. The runoff control methods, however, all cost more than Scheme 1. In addition, the roof disconnection method and the sewer separation method have some disadvantages. Therefore, the runoff control methods should be placed in lower preference than Schemes 1 and 2.

# 8.0 INTEGRATING CSO CONTROL AND FLOOD PROTECTION MEASURES

## 8.1 Basic Design Requirements

Although the study of basement flooding mitigation in the combined sewer area is outside the scope of this CSO study, a cursory analysis of the feasibility of integrating measures for basement flooding mitigation into CSO control schemes was carried out and presented in this section.

To facilitate discussion, the conditions for the design of the local sewers and the CSO control schemes are repeated in Table 8.1. The term "frequency" used in relation to a storm deserves some comments. For one and the same storm, the frequency of the storm may assume different values depending on which slice of the storm is being referred to. For example, consider a synthetic storm. Based on City of Toronto (1965) standard used in the City of York's original sewer separation program (Gore and Storrie Ltd., 1968), a 2-year synthetic storm has an intensity of 68.5 mm/hr if the storm duration is 15 minutes, but an intensity of 91.4 mm/hr if the duration is 8 minutes. Therefore, the storm intensity used in design can be varied substantially merely by varying the assumed storm duration. Consider a synthetic storm again from another point of view. A storm of a given amount of precipitation, say 20 mm, has a 3-year frequency if the storm duration is 15 minutes, but a 10-year frequency if the duration is 8 minutes. It is obvious that mentioning the frequency of a storm without specifying which slice of the storm is being referred to is ambiguous. This explains why the single real storm shown in Table 8.1(B) has different recurrence intervals. (Recurrence interval is the reciprocal of frequency).

One more preliminary point should be noted. Flooding arises when the rate of sewage flow exceeds the sewer capacity. The length of time for which the sewer capacity is exceeded has no relation to the occurrence of flooding. On the other hand, CSO control is the containment of a flow volume which is the product of the overflow rate and the overflow duration. In other words, the design conditions to be satisfied for flood protection and CSO control are different.

TABLE 8.1

BASIC DESIGN CRITERIA OF SEWERS AND CSO CONTROL

(A) Design of City of York Sewers (1)

| | Design Synthetic Storm | | | |
|---|---|---|---|---|
| | Reccurrence Interval (year) | Duration (minute) | Intensity (mm/hr) | Total Preci[illegible] (mm) |
| Existing Combined Sewers | 1.5 | 15 | 63.5 | 12.[illegible] |
| New Storm Sewers | 2.0 | 8 | 91.4 | 12.[illegible] |

(B) CSO Control Schemes 1, 2, 3A and 3B

| Characteristic of Real Storm (2) | | Reccurrence Interval (year) |
|---|---|---|
| Average Intensity: | 5.8 mm/hr | 1.8 |
| Max. Precip. in 1 hr: | 16.2 mm | 0.9 |
| Max. Precip. in 2 hr: | 16.4 mm | 0.7 |
| Total Event Precip.: | 17.4 mm | <0.3 |

Notes:

(1) Gore and Storrie Ltd., 1968.
(2) Storm producing largest CSO volume in April-October, 1979, excluding Event 790711.

We shall now examine the feasibility of integrating flooding protection measures into CSO control measures. We shall consider two storms, a 2-year storm and a 5-year storm.

## 8.2 Two-Year Storm

An 8-minute synthetic storm was assumed, following the City of York's practice. The work required for basement flooding mitigation and the CSO control schemes are shown in the upper half of Table 8.2. As expected, CSO Control Schemes 1 and 2 would not offer flood mitigation. In order to eliminate basement flooding, and assuming that local detention tanks are to be used for this purpose, Schemes 1 and 2 would each require an augmentation by local storage of 14,000 $m^3$. This local storage would not benefit CSO control because it would not reduce the required capacity of the CSO control schemes.

CSO Control Scheme 3A(i) (Local Detention Tanks) and Scheme 3A(iii) (Sewer Separation) would provide the required flooding protection. It may be questioned, however, whether it is justified to incur the extra cost by using Scheme 3A(i) or 3A(iii) instead of Scheme 1 or 2 for the sake of increasing the basement flooding protection from a recurrence interval of 1.5 years to 2.0 years.

Whether CSO Control Scheme 3A(ii) (Roof Leader Disconnection) would offer flood protection would depend on the soil moisture prevailing at the time of the storm. The maximum (dry soil) infiltration capacity of black loam (a common type of top soil in lawns) is about 60 mm/hr (Linsley, 1975). If the soil is saturated, the stormwater from roof leaders will not be able to infiltrate into the soil and an augmentation by local storage of 14,000 $m^3$ would be required.

## 8.3 Five-Year Storm

Again, an 8-minute synthetic storm was assumed. The work required for satisfying CSO control and basement flood protection are shown in the lower half of Table 8.2. The situation in this case is clear

TABLE 8.2

WORK REQUIRED FOR CSO CONTROL AND FLOOD PROTECTION

| Sewer Design Storm Considered | CSO Control Scheme Number | | Will CSO Scheme Control CSO ? | Will CSO Scheme Provide Flood Protection ? | Local Storage Additional To CSO Control for Flood Protection (1) |
|---|---|---|---|---|---|
| 2 - Year | 1. | Overflow Detention | Yes. Flow to down-stream restricted by sewer capacity. | No | 14,000 m^3 |
| | 2. | Resetting Regulators | Yes. Reason as above. | No | 14,000 m^3 |
| | 3A. | Local Detention | Yes. Excess flow stored locally. | Yes | 0 |
| | 3A. | Roof Dis-connection | Yes. Runoff not absorbed by soil flows overland. | Yes, if soil is dry. | 0 if soil is dry. 14,000 m^3 if soil is saturated. |
| | 3A. | Sewer Separation | Yes. | Yes | 0 |
| 5 - Year | 1. | Overflow Detention | Yes. As Scheme 1 above. | No | 28,000 m^3 |
| | 2. | Resetting Regulators | Yes. As Scheme 2 above. | No | 28,000 m^3 |
| | 3A. | Local Detention | Yes. Flow to down-stream restricted | No. 2-year storm only. | 14,000 m^3 |
| | 3A. | Roof Dis-connection | Yes. As roof dis-connection above. | No. Soil capa-city exceeded. | 14,000 m^3 if soil is dry. 28,000 m^3 if soil is saturated. |
| | 3A. | Sewer Separation | Yes. Flow to down-stream restricted by sewer capacity. | No. New storm sewers designed for 2-year storm. | 14,000 m^3 |

Note:
(1) For flood protection in runoff control areas assumed in Scheme 3A.

cut: basement flooding protection could be obtained only by control works in addition to the CSO control schemes. The function of the additional works was to reduce the flow rates to the capacities (for 1.5-year storm) of the existing sewers so that the sewers would not surcharge and basement flooding would not occur. The additional works would not reduce CSO because the flow put through by the additional works would not have a chance to flow through the sewers to the regulators if the additional works were not in place. The additional works in fact could increase CSO because that portion of the additional flow in excess of the threshold capacities of the regulators would overflow.

On the other hand, no works would be required for basement flooding protection if the duration of the 5-year storm was longer (say, 60 minutes) than the 8 minutes assumed in sewer design. In this case, the precipitation intensity would be 30.5 mm/hr only and the existing sewers would have sufficient capacities to transport the combined sewage. However, none of the CSO control schemes would have the capacity to contain the CSO from a precipitation of 30.5 mm falling in 1 hr because the precipitation was more than the 28.1 mm in 1 hr in Event 790711.

In conclusion, the measures required for CSO control and basement flooding protection in the City of York were distinctively different from each other and would not augment each other.

<u>References</u>

Acres Consulting Services Ltd., "TAWMS Technical Report No. 4: Humber River and Toronto Area Water Quality", September, 1984.

Braganza, A., Personal Communication, 1985.

Brandstetter, A., Field, R. and Torno, R., "Assessment of Mathemetical Models for Storm and Combined Sewer Management", U.S. Environment Protection Agency Report No. EPA-600/2-76-175a, August 1976.

Cermola, J. A. et al, "SWMM Application to Combined Sewerage in New Haven", <u>Journal of the Environmental Engineering Division</u>, ASCE, Vol. 105, No. EE6, December 1979, pp. 1035-1048.

City of Etobicoke, Informal Communications with the City of Etobicoke: Internal Inflow/Infiltration Study Reports, 1983.

Colyer, P.J., "Detention Storage As An Equivalent Rainfall Depth", Proceedings of Conference on Stormwater Detention Facilities, ASCE, August 1982, pp. 86-93.

Crozier, T. et al "SAREMP, Municipal Experience in Inflow Control Through Removal of Household Roof Leeders", January 1984.

Dorsch Consult Ltd (Toronto), "Study on Operational Behaviour of Principal City Interceptor Sewers", Vol. 1 and 2, October 1979.

Finnemore, E.J., "Stormwater Pollution Control: Structural Measures", <u>Journal of the Environmental Engineering Division</u>, ASCE, Vol. 108, No. EE4, August 1982, pp. 706-721.

Fortin, M., "Cost-Effectiveness Analysis Procedures for TAWMS", February, 1985.

Gartner Lee Associates Ltd, "Storm, Sanitary and Combined Sewer Mapping and Data Enumeration", July 1983.

Gore & Storrie Ltd., "Borough of York: Report on the Central Area Sewer System", November 1968.

Gore & Storrie Ltd., "City of Cornwall: Report on Combined Sewer System Analysis", July 1980.

Gore & Storrie Ltd., and Proctor & Redfern Group, "Report on Rideau River Stormwater Management Study, Phase I", April 1981.

Gore & Storrie Ltd, "Humber River Bacteriological Study", January 1984 (Draft)

Holbrook, R.F. et al, "Stormwater Studies and Alternatives in Atlanta", Journal of the Environmental Engineering Division, ASCE, Vol. 102, No. EE6, December 1976, pp. 1263-1277.

Huber, W.C. et al, "Storm Water Management Model User's Manual, Version III", U.S. Environmental Protection Agency, Municipal Environmental Research Laboratory, Cincinnati, November, 1981.

James, W. "Development of A Stormwater Overflow Model for Pollutional Loadings to Hamilton's Receiving Waters", May 1980.

Kronis, H. et al, "Full-Scale Evaluation of High Rate Screening Devices for Treatment of Sanitary Sewage By-Pass Flow", Canada-Ontario Agreement on Great Lakes Water Quality Research Report No. 103, Date 1975 (?)

Kronis, H., "Physical-Chemical Treatment and Disinfection of Stormwater", Research Report No. 88, Ontario Ministry of the Environment, March 1982.

Kuribayashi, M. et al, "Challenging Combined Sewer Problems in Japan", Journal of Water Pollution Control, Vol. 52, No. 5, May 1980, pp. 890-898.

Linsley, R.K. et al, "Hydrology for Engineers", 2nd edition, McGraw Hill, Inc., 1975.

Marshall, Macklin, Monaghan Ltd., "Kennedy Burnett Modelling Study, Objective 1", January 1982.

Meinholz, T.L. et al, "Verification of the Water Quality Impacts of Combined Sewer Overflow", U.S. Environmental Protection Agency Report No. EPA-600/2-79-155, December 1979.

Mendenhall, W. and Reinmuth, J.E., "Statistics for Management and Economics", 3rd edition, Duxbury Press, 1978.

Metropolitan Toronto, Municipality of, Letter Addressed to Ministry of the Environment, February 1984.

Ministry of the Environment (Ontario), "TAWMS Interim Report on Toronto Area Water Quality", April 1983.

Ministry of the Environment, "Guidelines for the Design of Sanitary Sewage Systems", Environmental Approvals and Project Engineering Branch, July 1984.

(NCASI) National Council of the Paper Industry for Air and Stream Improvement Inc., "A Study of the Selection, Calibration and Verification of Mathematical Water Quality Models", Technical Bulletin No. 367, March, 1982.

Nebolsine, R. et al, "High Rate Filtration of Combined Sewer Overflows", U.S. Environmental Protection Agency Water Pollution Control Research Series 11023 EYI 04/72, April 1972.

Pitt, R., "Demonstration of Non-point Pollution Abatement through Improved Street Cleaning Practices", U.S. Environmental Protection Agency, Report No. EPA-600/2-79-161, August 1979.

Pitt, R. et al, "TAWMS Humber River Pilot Watershed Project", (Draft Report), November 1984.

Prah, D.H. et al "Combined Sewer Overflow Treatment by Screening and Terminal Ponding", U.S. Environmental Protection Agency Report No. EPA-600/2-79-086, August 1979.

Robinson, M. and James, W., "Continuous SWMM Quality Modelling for the City of Hamiltgn Using AES Data", Proceedings, Stormwater and Water Quality Management Modelling and SWMM User Group Meeting", September 1981, pp. 469-491.

Storm Drainage Research Committee, John Hopkins University, "The Design of Storm-Water Inlets", June 1956, pp. 192.

Talhami, A., "Temporary Detention Cuts Storm Flow Peaks", Civil Engineering, ASCE, Vol. 50, No. 12, December 1980, pp. 72-75.

Vatagoda, S. et al, "Impact of Roof Leader Disconnection Practices on Basement Flooding", Proceedings 1982 (SCE) Conference, Edmonton, Alberta.

Waller, D.H. et al, "Winter Runoff from An Urban Catchment", Canada-Ontario Agreement on Great Lakes Water Quality Research Report No. 41, 1974(?)

Whipple, W. Jr. et al, "Effects of Storm Frequency on Pollution from Urban Runoff", Journal of the Water Pollution Control Federation, Vol. 49, No. 11, November 1977, pp. 2243-2248.

Whipple, W., "Discussion Summary", Proceedings of Conference on Stormwater Detention Facilities, ASCE, August 1982, pp. 267-270.

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## APPENDICES

## APPENDIX A1

## CATCHMENT DATA

WASTEWATER PRODUCTION RATES (1)

Apartments

Reference: Table 3 for Years 1980-82

Avg. consumption = 586,581 gal/d = 2,666,597 l/d

Total population
= 2.3 persons/units (Table 2) x 3,475 units (Table 3)
= 7,993 persons

Avg. consumption = 2,666,597/7,993 = 333.6 lpcd

Offices

Reference: Table 4 for Years 1980-82

Land area: floor area = 569 ac: 13,644,000 sq ft
= 0.0417 ac/1,000 sq ft

Avg. consumption = 137 gal/0.0417 ac/d
= 36,905 l/ha/d

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Note:

(1) All data from "North York: Summary of City-Wide Water Consumption Statistics (1976)-updated to 1982", except low/medium residential water consumption. The latter data were from Water Accounts Dept., City of Etobicoke, November 1983.

Shopping Centres

Total land area = 175.7 ac

Total avg. consumption = 290,997 gal/d = 18,604 l/ha/d

Commercial Consumption

Ratio of office land: shopping land = 569: 1,328 (Table 1)

Land Use Weighted Commercial Consumption
= 18,604 x 1,328 + 36,905 x 569 / (1,328 + 569) = 24,093 l/ha/d

Industrial Consumption

Avg. consumption on net area = 3,808 gal/ac/d

Vacant land = 26%

Avg. consumption on gross area = 3,808 x (100 - 26) / 100 gal/ac/d
= 31,653 l/ha/d

Low/Medium Residential Consumption

Avg. consumption per household in 124 days = 96,795 l

4.4 persons per household (North York Revised Table 2)

Avg. consumption = 96,800/4.4/124 = 177 lpcd

TABLE A 1.1

CATCHMENT DATA (Sheet 1 of 7) *
(A) COMBINED SEWER CATCHMENTS

| Catchment I.D. No. (Gartner-Lee) | Catchment I.D. No. (CSO Model) | Catchment Name (If Any) | Land Use | Area (ha) | Population | WP (m3/d) | % Imper-viousness | Overland Flow Length (m) | Road Length (km) |
|---|---|---|---|---|---|---|---|---|---|
| 6.2.05 | 2531 | | 1 | 135.1 | | 2,564.1 | 18.9 | | |
| | 2532 | | 2 | 0.9 | | 56.9 | 50.0 | | |
| (Excluding | 2533 | Kitchener | 3 | 18.4 | | 443.4 | 100.0 | | |
| Sanitary | 2534 | Ave | 4 | 21.7 | | 778.3 | 42.0 | | |
| Areas) | 2535 | | 5 | 22.2 | | 6.8 | 5.0 | | |
| | 2536 | | 1 | 32.1 | | | 18.9 | | |
| | | Sub-Total | | 230.4 | 14,656 | 3,849.4 | | 2,304.0 | 23.9 |
| | 2541 | | 1 | 301.8 | | 5,727.4 | 18.9 | | |
| | 2542 | | 2 | 2.0 | | 127.0 | 50.0 | | |
| | 2543 | "Remainder" | 3 | 41.1 | | 990.5 | 100.0 | | |
| | 2544 | | 4 | 48.5 | | 1,738.5 | 42.0 | | |
| | 2545 | | 5 | 49.6 | | 15.1 | 5.0 | | |
| | 2546 | | 1 | 71.7 | | | 18.9 | | |
| | | Sub-Total | | 514.7 | 32,740 | 8,598.5 | | 5,147.0 | 53.6 |
| | 2551 | | 1 | 80.8 | | 1,532.5 | 18.9 | | |
| | 2552 | | 2 | 0.5 | | 34.0 | 50.0 | | |
| | 2553 | Keele | 3 | 11.0 | | 265.0 | 100.0 | | |
| | 2554 | | 4 | 13.0 | | 465.2 | 42.0 | | |
| | 2555 | | 5 | 13.3 | | 4.0 | 5.0 | | |
| | 2556 | | 1 | 19.2 | | | 18.9 | | |
| | | Sub-Total | | 137.8 | 8,761 | 2,300.7 | | 1,378.0 | 14.3 |

* See notes at end of table.

WP = Wastewater production rate.

TABLE A 1.1

CATCHMENT DATA (Sheet 2 of 7)
(A) COMBINED SEWER CATCHMENTS

| Catchment I.D. No. (Gartner-Lee) | Catchment I.D. No. (CSO Model) | Catchment Name (If Any) | Land Use | Area (ha) | Population | WP (m3/d) | % Imperviousness | Overland Flow Length (m) | Road Length (km) |
|---|---|---|---|---|---|---|---|---|---|
| 6.2.15 | 2151 | | 1 | 101.5 | | 1,364 | 33.0 | | |
| (Excluding | 2152 | | 2 | 4.0 | | 203 | 50.0 | | |
| Sanitary | 2153 | Mount | 3 | 14.5 | | 349 | 100.0 | | |
| Areas) | 2154 | Dennis | 4 | 34.0 | | 1,079 | 90.0 | | |
| | 2155 | | 5 | 13.0 | | 0 | 10.0 | | |
| | | Sub-Total | | 167.0 | 8,314 | 2,995 | | 2,451 | 17.4 |
| 6.2.18 | 2181 | | 1 | 149.8 | | 1,769 | 28.0 | | |
| | 2184 | Rockcliffe | 4 | 24.5 | | 776 | 95.0 | | |
| | 2185 | | 5 | 21.8 | | 8 | 2.0 | | |
| | | Sub-Total | | 196.1 | 9,994 | 2,553 | | 1,900 | 16.1 |
| Total of (A) | | | | 1,246.0 | 74,465 | 20,297 | | 13,180 | 125.3 |

TABLE A 1.1

CATCHMENT DATA (Sheet 3 of 7)
(B) SANITARY SEWER CATCHMENTS

| Catchment I.D. No. (Gartner-Lee) | Inlet I.D. No. (CSO Model) | Catchment Name (If Any) | Land Use | Area (ha) | Population | WP (m3/d) |
|---|---|---|---|---|---|---|
| 3.1.07 | 42 | | 1 | 268.4 | | 1,969 |
| | | | 2 | 28.1 | | 1,513 |
| | | | 3 | 38.3 | | 923 |
| | | | 4 | 700.8 | | 22,183 |
| | | | 5 | 173.0 | | 60 |
| | | Sub-Total | | 1,208.6 | 15,655 | 26,588 |
| 3.1.08 | 21 | | 1 | 718.2 | | 6,271 |
| | | | 2 | 60.0 | | 3,593 |
| | | | 3 | 56.8 | | 1,368 |
| | | | 4 | 444.0 | | 14,054 |
| | | | 5 | 405.8 | | 103 |
| | | Sub-Total | | 1,684.8 | 46,190 | 25,389 |
| 3.1.09 | 14 | | 1 | 510 | | 2,990 |
| | | | 2 | 61 | | 2,549 |
| | | | 3 | 14 | | 347 |
| | | | 4 | 38 | | 1,225 |
| | | | 5 | 319 | | 103 |
| | | Sub-Total | | 944.5 | 24,527 | 7,214 |
| 3.1.10 | 2 | | 1 | 483.1 | | 3,611 |
| | | | 2 | 56.2 | | 3,007 |
| | | | 3 | 28.0 | | 646 |
| | | | 4 | 7.1 | | 225 |
| | | | 5 | 238.4 | | 78 |
| | | Sub-Total | | 812.8 | 29,402 | 7,567 |

TABLE A 1.1

CATCHMENT DATA (Sheet 4 of 7)
(B) SANITARY SEWER CATCHMENTS

| Catchment I.D. No. (Gartner-Lee) | Inlet I.D. No. (CSO Model) | Catchment Name (If Any) | Land Use | Area (ha) | Population | WP (m3/d) |
|---|---|---|---|---|---|---|
| 3.1.11 | 50 | | 1 | 596.1 | | 2,902 |
| | | | 2 | 43.7 | | 1,432 |
| | | | 3 | 81.0 | | 1,952 |
| | | | 4 | 302.9 | | 9,588 |
| | | | 5 | 706.3 | | 233 |
| | | Sub-Total | | 1,730.0 | 20,680 | 16,107 |
| 3.1.12 | 6 (3) | | 1 | 369.0 | | 2,625 |
| | | | 2 | 27.8 | | 1,452 |
| | | | 3 | 79.8 | | 1,923 |
| | | | 4 | 351.1 | | 11,113 |
| | | | 5 | 486.0 | | 112 |
| | | Sub-Total | | 1,313.7 | 19,179 | 17,225 |
| 3.1.13 | 6 | | 1 | 634.7 | | 3,690 |
| | | | 2 | 20.3 | | 885 |
| | | | 3 | 23.8 | | 573 |
| | | | 5 | 75.8 | | 26 |
| | | Sub-Total | | 754.6 | 23,499 | 5,174 |
| 3.1.14 | 94 | | 1 | 400.8 | | 2,346 |
| | | | 2 | 15.1 | | 644 |
| | | | 3 | 47.7 | | 1,149 |
| | | | 4 | 207.3 | | 6,562 |
| | | | 5 | 258.9 | | 86 |
| | | Sub-Total | | 929.8 | 15,185 | 10,787 |

TABLE A 1.1

CATCHMENT DATA (Sheet 5 of 7)
(B) SANITARY SEWER CATCHMENTS

| Catchment I.D. No. (Gartner-Lee) | Inlet I.D. No. (CSO Model) | Catchment Name (If Any) | Land Use | Area (ha) | Population | WP (m3/d) |
|---|---|---|---|---|---|---|
| 3.1.16 | 94 | | 1 | 205.7 | | 2,088 |
| | | | 2 | 38.9 | | 2,952 |
| | | | 3 | 34.1 | | 822 |
| | | | 4 | 225.0 | | 7,122 |
| | | | 5 | 72.0 | | 17 |
| | | Sub-Total | | 575.7 | 20,635 | 13,001 |
| 3.1.17 | 94 | | 1 | 241.4 | | 1,777 |
| | | | 2 | 21.8 | | 1,083 |
| | | | 3 | 95.2 | | 2,294 |
| | | | 4 | 492.4 | | 15,586 |
| | | | 5 | 167.6 | | 34 |
| | | Sub-Total | | 1,018.4 | 13,280 | 20,774 |
| 3.1.20 (4) | | | | | | |
| 4.1.02 | 120 | | 1 | 990.7 | | 13,057 |
| | | | 2 | 201.2 | | 13,273 |
| | | | 3 | 84.1 | | 2,026 |
| | | | 4 | 340.8 | | 14,578 |
| | | | 5 | 868.5 | | 259 |
| | | Sub-Total | | 2,485.5 | 113,509 | 39,403 |
| 4.1.03 | 150 | | 1 | 747.1 | | 6,486 |
| | | | 2 | 52.5 | | 2,949 |
| | | | 3 | 46.4 | | 1,118 |
| | | | 4 | 441.9 | | 13,987 |
| | | | 5 | 258.5 | | 60 |
| | | Sub-Total | | 1,546.4 | 45,472 | 24,600 |

TABLE A 1.1

CATCHMENT DATA (Sheet 6 of 7)
(B) SANITARY SEWER CATCHMENTS

| Catchment I.D. No. (Gartner-Lee) | Inlet I.D. No. (CSO Model) | Catchment Name (If Any) | Land Use | Area (ha) | Population | WP (m3/d) |
|---|---|---|---|---|---|---|
| 4.1.06 | 50 | | 1 | 573.7 | | 6,351 |
| | | | 2 | 55.3 | | 3,445 |
| | | | 3 | 18.4 | | 443 |
| | | | 4 | 666.8 | | 21,106 |
| | | | 5 | 496.1 | | 156 |
| | | Sub-Total | | 1,810.3 | 46,199 | 31,501 |
| 5.1.01 | 96 | | 1 | 149.0 | | 1,356 |
| | | | 2 | 12.4 | | 852 |
| | | | 3 | 6.3 | | 152 |
| | | | 4 | 7.5 | | 321 |
| | | | 5 | 238.2 | | 78 |
| | | Sub-Total | | 413.4 | 10,210 | 2,676 |
| 5.1.19 | 220 | | 1 | 5.0 | | 83 |
| | | | 3 | 0.9 | | 22 |
| | | | 4 | 26.9 | | 1,083 |
| | | | 5 | 6.2 | | |
| | | Sub-Total | | 39.0 | 471 | 1,188 |
| 5.1.21 | 2 | | 1 | 10.5 | | 295 |
| | | | 3 | 4.0 | | 96 |
| | | | 4 | 0.5 | | 16 |
| | | | 5 | 0.5 | | 0 |
| | | Sub-Total | | 15.5 | 1,669 | 407 |
| 6.1.04 | 10 | | 1 | 39.9 | | 238 |
| | | | 2 | 21.6 | | 609 |
| | | | 3 | 1.0 | | 24 |
| | | | 5 | 63.7 | | 17 |
| | | Sub-Total | | 126.2 | 3,169 | 888 |

TABLE A 1.1

CATCHMENT DATA
(B) SANITARY SEWER CATCHMENTS (Sheet 7 of 7)

| Catchment I.D. No. (Gartner-Lee) | Inlet I.D. No. (CSO Model) | Catchment Name (If Any) | Land Use | Area (Ha) | Population | WP (m3/d) |
|---|---|---|---|---|---|---|
| 6.2.05 | 365 | | 1 | 41.9 | | 694 |
| | | | 2 | 10.3 | | 548 |
| | | | 3 | 1.5 | | 36 |
| | | | 4 | 11.0 | | 471 |
| | | | 5 | 2.5 | | 26 |
| | | Sub-Total | | 67.2 | 5,559 | 1,304 |
| 6.2.15 | 420 | | 1 | 15.5 | | 194 |
| | | | 2 | 1.0 | | 51 |
| | | Sub-Total | | 16.5 | 1,250 | 245 |

| | | | |
|---|---|---|---|
| Total of (B) | 17,502.8 | 455,740 | 252,038 |
| Total of (A) | 1,246.0 | 74,465 | 20,297 |
| TOTALS OF (A) + (B) | 18,748.8 | 530,205 | 272,335 |

Notes :
(1) TWP = Theoretical waste water production rate.
(2) Land uses:
1 = Residual, low density.
2 = Residual, medium/high density.
3 = Commercial.
4 = Industrial.
5 = Open space and miscellaneous.
(3) Input of 17,280 m3/d from Mississauga not shown.
(4) Flow goes to Lakeview WPCP.

# APPENDIX B1

# INSTRUMENTATION FOR DATA COLLECTION

## INSTRUMENTATION AND SAMPLE COLLECTION

ISCO 2500 series flow monitors were used in all stations except the inlet to the Hyde Ave tank and Station 5. These monitors are automatic computerized instrument recording the depths of flow, and were set to record at 5-minute intervals continuously. The depth sensors are small, streamlined submersible pressure transducers and were attached to the invert of the sewer or the overflow weir crest. The data collected were saved on a computer tape and subsequently transferred to micro-computer diskettes and processed by a micro-computer. Since there was no accessible location to monitor the inlet to the Hyde Ave tank, data for this location had to be obtained from the tank's permanent bubbler water level recorder. This instrument records on a weekly chart. To improve the time resolution of the time base of the recording, a second pressure signal recorder having a higher chart speed was installed. Despite this effort, however, the data for this station did not have as a high resolution as the ISCO stations' because the instrument read the depth of water in the tank (not in the sewer) and one graduation of the chart was 50 mm of the tank depth.

Due to persistent high flow in the sewer, flow data at Station 5 was measured with an N-Con surface tracking monitor, which is an analog instrument recording the flow depth on a paper chart continuously. The sewage surface is continuously tracked by a suspended probe linked by a mechanism to the recording pen.

The ISCO monitor worked satisfactorily for most of the time. On occasions it malfunctioned, mostly because of the seal failure on

the pressure transducers. The manufacturer replaced the faulty sensors under warranty.

For sample collection, ISCO model 2100 automatic samplers were used. Each sampler contained 24 sample bottles of 1-litre size. All of the monitoring gear is suitable for installation inside a manhole. Sample was collected via a suction tube and the suction pump was driven by battery. The sampler was actuated in two ways. On overflow sampling, float switches were used to signal the sampler when sewage passed over the weir. For sampling in the sewer line itself, the start time of the sampling cycle was pre-set and samples were taken at fixed time intervals. Samples and flow rates were time correlated based on known time setting.

In this project, DWF samples were 24-hour composite samples. For combined sewage, sequential samples were collected as data were needed for deriving a relationship of flow concentrations with flow rates (load rate curve) for later use with the simulation model. In a storm event, a number of sequential samples were first collected from Site 3. After the storm passed over, the event precipitation was examined to see if the samples should be analyzed by the laboratory. The aim of the sampling program was to collect data for 8 to 10 events and that the events should be approximately evenly distributed among the precipitation groups of 5 to 10 mm; 10 to 15 mm and over 15 mm. If analysis was justified, the hydrograph of the event was used to guide the selection of samples such that samples at the time of major crests and troughs of the hydrograph was picked. The selection was necessary to reduce the number of samples to be analyzed and yet to provide as much information about changes in pollutant concentrations as possible. About 6 to 8 samples were analyzed for a selected event.

Precipitation was measured with a tipping bucket rain guage and a standard guage, following Atmospheric Environment Services practices. The data were discretized into 5-minute intervals.

# APPENDIX B2

## FLOW QUANTITY AND QUALITY DATA

TABLE B 2.1

OBSERVED DRY WEATHER FLOW AT HILLARY AVE. SEWER CATCHMENT
(STATION I.D. No. 1) (Sheet 1 of 2)

(A) Dates of Data

| | Week 1 | Week 2 | Week 3 | Week 4 |
|---|---|---|---|---|
| Mon | 821115 | 830613 | 830620 | No Data |
| Tue | 821119 | 830510 | 830517 | 830524 |
| Wed | 821117 | 830511 | 830518 | 830601 |
| Thu | 821028 | 830428 | 830512 | 830602 |
| Fri | 821029 | 830429 | 830513 | 830527 |
| Sat | 821030 | 830528 | 830611 | 830618 |
| Sun | 821107 | 830605 | 830612 | 830619 |

(B) Average Daily Volume ($m^3/d$)

| Week 1 | Week 2 | Week 3 | Week 4 | 4-Week Avg. | Standard Deviation |
|---|---|---|---|---|---|
| 25,970 | 27,775 | 26,768 | 27,270 | 26,946 | 770 |

(C) Ratios of Daily Flow Variations

| | Week 1 | Week 2 | Week 3 | Week 4 | 4-Week Avg | Standard Deviation |
|---|---|---|---|---|---|---|
| Mon | .89 | 1.02 | 1.00 | No Data | 1.01 | 0.07 |
| Tue | .74 | .99 | .98 | 1.10 | .95 | 0.15 |
| Wed | .85 | .99 | .98 | .99 | .95 | 0.07 |
| Thu | 1.23 | 1.11 | 1.03 | .98 | 1.09 | 0.11 |
| Fri | 1.27 | .91 | 1.05 | 1.02 | 1.06 | 0.15 |
| Sat | 1.27 | 1.00 | .99 | .99 | 1.06 | 0.14 |
| Sun | .74 | .99 | .96 | .92 | .90 | 0.11 |

TABLE B2.1 (Sheet 2 of 2)

(D) Ratios of Hourly Flow Variations

| Hour | Week 1 | Week 2 | Week 3 | Week 4 | 4-Week Avg | Standard Deviation |
|---|---|---|---|---|---|---|
| 1 | .86 | .68 | .69 | .72 | .74 | .08 |
| 2 | .72 | .59 | .61 | .62 | .64 | .06 |
| 3 | .63 | .55 | .57 | .59 | .59 | .03 |
| 4 | .60 | .54 | .55 | .58 | .57 | .03 |
| 5 | .57 | .62 | .58 | .59 | .59 | .02 |
| 6 | .59 | .75 | .72 | .74 | .70 | .07 |
| 7 | .71 | .95 | .98 | .98 | .91 | .13 |
| 8 | .96 | 1.15 | 1.21 | 1.20 | 1.13 | .12 |
| 9 | 1.20 | 1.21 | 1.26 | 1.25 | 1.23 | .03 |
| 10 | 1.25 | 1.23 | 1.28 | 1.26 | 1.26 | .02 |
| 11 | 1.26 | 1.22 | 1.27 | 1.25 | 1.25 | .02 |
| 12 | 1.26 | 1.18 | 1.22 | 1.20 | 1.22 | .03 |
| 13 | 1.20 | 1.13 | 1.16 | 1.14 | 1.16 | .03 |
| 14 | 1.16 | 1.08 | 1.11 | 1.10 | 1.11 | .03 |
| 15 | 1.11 | 1.04 | 1.07 | 1.06 | 1.07 | .03 |
| 16 | 1.07 | 1.02 | 1.05 | 1.06 | 1.05 | .02 |
| 17 | 1.06 | 1.05 | 1.08 | 1.08 | 1.07 | .02 |
| 18 | 1.10 | 1.58 | 1.17 | 1.14 | 1.25 | .22 |
| 19 | 1.18 | 1.34 | 1.19 | 1.18 | 1.22 | .08 |
| 20 | 1.22 | 1.12 | 1.14 | 1.13 | 1.15 | .05 |
| 21 | 1.16 | 1.07 | 1.11 | 1.09 | 1.11 | .04 |
| 22 | 1.10 | 1.06 | 1.09 | 1.10 | 1.09 | .02 |
| 23 | 1.04 | .99 | 1.03 | 1.03 | 1.02 | .02 |
| 24 | 1.00 | .85 | 0.88 | 0.89 | 0.91 | .07 |

TABLE B 2.2

OBSERVED DRY WEATHER FLOW AT BLACK CREEK STS.
(STATION I.D. No. 5) (Sheet 1 of 2)

---

(A) Dates of Data

| | Week 1 | Week 2 |
|---|---|---|
| Mon | 830620 | 830711 |
| Tue | 830621 | 830712 |
| Wed | 830706 | 830713 |
| Thu | 830707 | 830714 |
| Fri | 830624 | 830708 |
| Sat | 830702 | 830813 |
| Sun | 830703 | 830710 |

(B) Average Daily Volume (m^3/d)

| Week 1 | Week 2 |
|---|---|
| 79,253 | 93,984 |

(C) Ratios of Daily Flow Variations

| | Week 1 | Week 2 | 2-Week Avg. |
|---|---|---|---|
| Mon | .95 | .79 | .87 |
| Tue | .93 | 1.13 | 1.03 |
| Wed | 1.03 | 1.27 | 1.15 |
| Thu | .94 | 1.28 | 1.11 |
| Fri | .98 | .80 | .89 |
| Sat | 1.06 | 1.13 | 1.10 |
| Sun | 1.11 | .61 | .86 |

TABLE B2.2 (Sheet 2 of 2)

(D) Ratios of Hourly Flow Variations

| Hour | Week 1 | Week 2 | 2-Week Aug. |
|---|---|---|---|
| 1 | .84 | .82 | .83 |
| 2 | .67 | .65 | .66 |
| 3 | .54 | .51 | .53 |
| 4 | .48 | .47 | .47 |
| 5 | .46 | .46 | .46 |
| 6 | .48 | .49 | .49 |
| 7 | .66 | .65 | .66 |
| 8 | .93 | .89 | .91 |
| 9 | 1.13 | 1.09 | 1.11 |
| 10 | 1.20 | 1.17 | 1.18 |
| 11 | 1.32 | 1.25 | 1.28 |
| 12 | 1.36 | 1.39 | 1.38 |
| 13 | 1.35 | 1.34 | 1.34 |
| 14 | 1.30 | 1.28 | 1.29 |
| 15 | 1.21 | 1.16 | 1.18 |
| 16 | 1.11 | 1.16 | 1.13 |
| 17 | 1.07 | 1.16 | 1.12 |
| 18 | 1.13 | 1.18 | 1.15 |
| 19 | 1.23 | 1.27 | 1.25 |
| 20 | 1.24 | 1.31 | 1.26 |
| 21 | 1.14 | 1.18 | 1.16 |
| 22 | 1.07 | 1.04 | 1.05 |
| 23 | 1.08 | 1.04 | 1.06 |
| 24 | 1.00 | 1.00 | 1.00 |

TABLE B2.3 (Sheet 1 of 2)

OBSERVED DRY WEATHER FLOW QUALITIES

| Sampling Date | (Concentration (mg/l) at Hillary Ave. Catchment) BOD5 | RSP | PPUT | PPO4FR | CUUT | PBUT | ZNUT | Sampling Date | CDUT* Conc. At Mt. Dennis/ Rockcliffe |
|---|---|---|---|---|---|---|---|---|---|
| 821103 | 218.0 | 160.0 | 3.65 | 1.78 | 0.14 | 0.09 | 0.31 | 830216 | 0.01 (Rock) |
| 821105 | 137.0 | 154.0 | 1.45 | 0.86 | 0.07 | 0.08 | 0.33 | 830223 | 0.01 (Rock) |
| 821106 | 400.0 | 330.0 | 6.20 | 3.70 | 0.47 | 0.07 | 0.47 | 830224 | 0.02 (MT.D) |
| 821109 | 265.0 | 299.0 | 6.50 | 3.35 | 0.15 | 0.08 | 0.25 | 830224 | 0.01 (Rock) |
| 821110 | 187.0 | 180.0 | 4.45 | 2.34 | 0.12 | 0.03 | 0.12 | 830228 | 0.01 (MT.D) |
| 821117 | 199.0 | 160.0 | 4.47 | 2.32 | 0.13 | 0.03 | 0.16 | 830301 | 0.01 (MT.D) |
| 821118 | 200.0 | 237.0 | 5.10 | 2.54 | 0.10 | 0.04 | 0.19 | 830301 | 0.03 (MT.D) |
| 821120 | 230.0 | 347.0 | 6.25 | 3.38 | 0.17 | 0.08 | 0.19 | 830303 | 0.02 (MT.D) |
| 821121 | 184.0 | 421.0 | 12.20 | 2.40 | 0.21 | 0.13 | 0.26 | 830303 | 0.01 (Rock) |
| 821122 | 135.0 | 249.0 | 4.25 | 2.22 | 0.09 | 0.04 | 0.13 | 830304 | 0.02 (MT.D) |
| 821123 | 143.0 | 162.0 | 5.15 | 1.70 | 0.09 | 0.03 | 0.23 | 830304 | 0.01 (Rock) |
| Average | 208.9 | 245.4 | 5.42 | 2.42 | 0.16 | 0.06 | 0.24 | | 0.01 |
| Standard Deviation | 75.1 | 92.4 | 2.66 | 0.83 | 0.11 | 0.03 | 0.10 | | 0.007 |

Notes:
RSP = Suspended Solids
CUUT = Copper
CDUT = Cadmium in mg/l
ZNUT = Zinc
PBUT = Lead
CDUT* Conc. in mg/l
PPO4FR = Filtered, Reactivi Phosphorous
PPUT = Total Phosphorous

TABLE B2.3 (Sheet 2 of 2)

OBSERVED DRY WEATHER FLOW QUALITIES

| Sampling Date | (Concentration (mg/l) at Black Creek Trunk Sewer) BOD5 | RSP | PPUT | PPO4FR | CUUT | PBUT | ZNUT | Sampling Date | CDUT* Conc. At Mt. Dennis/ Rockcliffe |
|---|---|---|---|---|---|---|---|---|---|
| 821102 | 275.0 | 151.0 | 5.00 | 2.38 | 0.43 | 0.11 | 0.13 | 830216 | 0.01 (Rock) |
| 821103 | 208.0 | 155.0 | 3.80 | 1.76 | 0.48 | 0.06 | 0.23 | 830223 | 0.01 (Rock) |
| 821105 | 186.0 | 114.0 | 1.62 | 1.58 | 0.32 | 0.06 | 0.12 | 830224 | 0.02 (MT.D) |
| 821106 | 240.0 | 150.0 | 14.80 | 1.40 | 0.36 | 0.04 | 0.16 | 830224 | 0.01 (Rock) |
| 821107 | 157.0 | 153.0 | 3.45 | 1.50 | 0.18 | 0.02 | 0.11 | 830228 | 0.01 (MT.D) |
| 821108 | 280.0 | 303.0 | 4.62 | 2.84 | 0.28 | 0.04 | 0.15 | 830301 | 0.01 (MT.D) |
| 821109 | 177.0 | 177.0 | 4.50 | 4.40 | 0.50 | 0.05 | 0.20 | 830301 | 0.03 (MT.D) |
| 821117 | 234.0 | 175.0 | 6.10 | 2.32 | 0.64 | 0.03 | 0.22 | 830303 | 0.02 (MT.D) |
| 821118 | 230.0 | 156.0 | 6.20 | 2.38 | 0.47 | 0.03 | 0.15 | 830303 | 0.01 (Rock) |
| 821119 | 143.0 | 145.0 | 5.50 | 2.62 | 0.37 | 0.02 | 0.15 | 830304 | 0.02 (MT.D) |
| 821120 | 134.0 | 180.0 | 6.10 | 3.96 | 0.34 | 0.02 | 0.14 | 830304 | 0.01 (Rock) |
| 821121 | 150.0 | 217.0 | 5.45 | 3.38 | 0.20 | 0.03 | | | |
| 821122 | 127.0 | 190.0 | 5.75 | 2.84 | 0.18 | 0.03 | 0.14 | | |
| 821123 | 146.0 | 168.0 | 6.25 | 2.76 | 0.31 | 0.05 | 0.28 | | |
| Average | 191.9 | 173.8 | 5.66 | 2.58 | 0.37 | 0.04 | 0.17 | | 0.01 |
| Standard Deviation | 52.4 | 44.3 | 2.94 | 0.89 | 0.13 | 0.02 | 0.05 | | 0.007 |

Notes:
RSP = Suspended Solids    ZNUT = Zinc    PPO4FR = Filtered, Reactivi Phosphorous

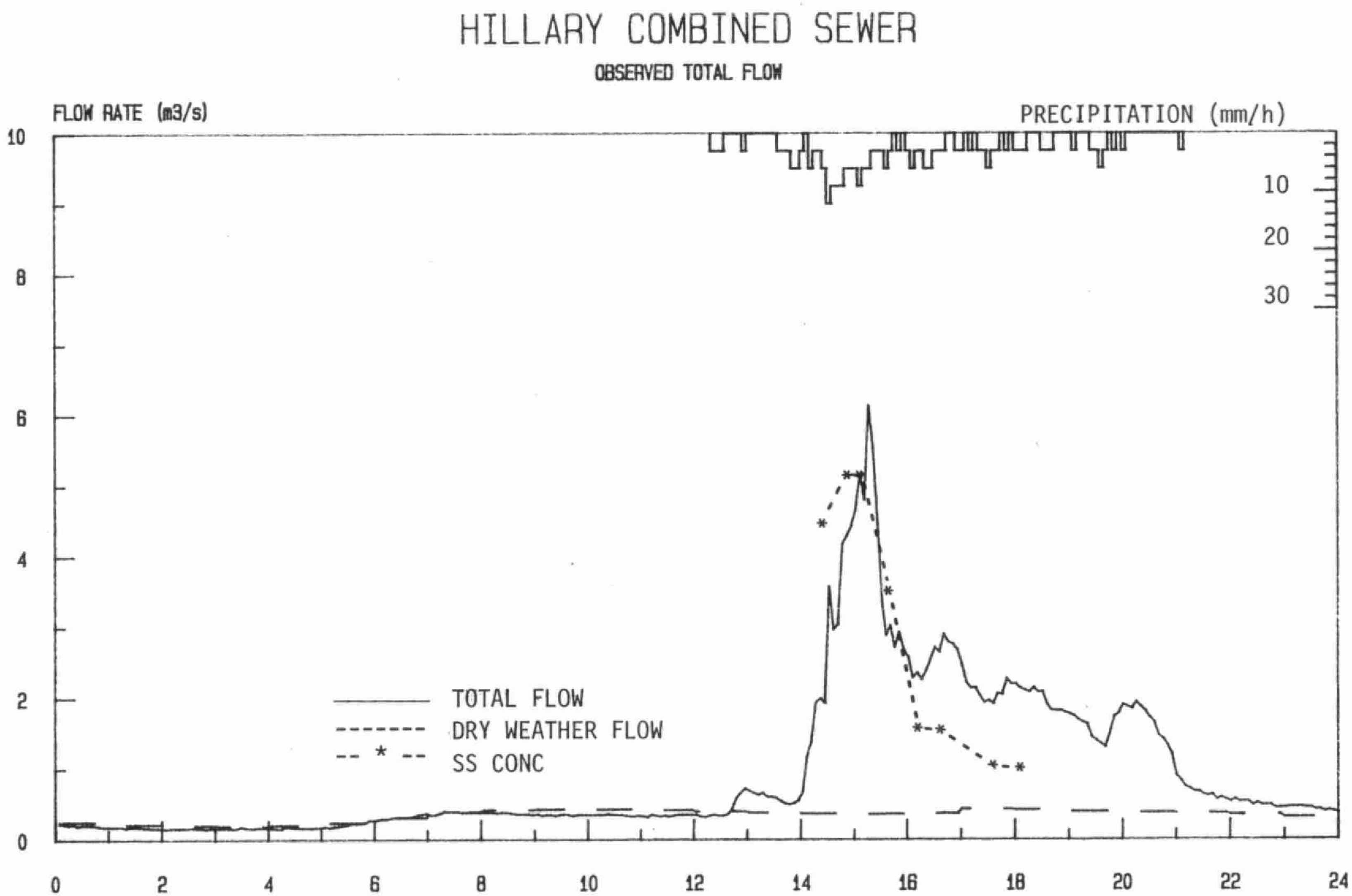
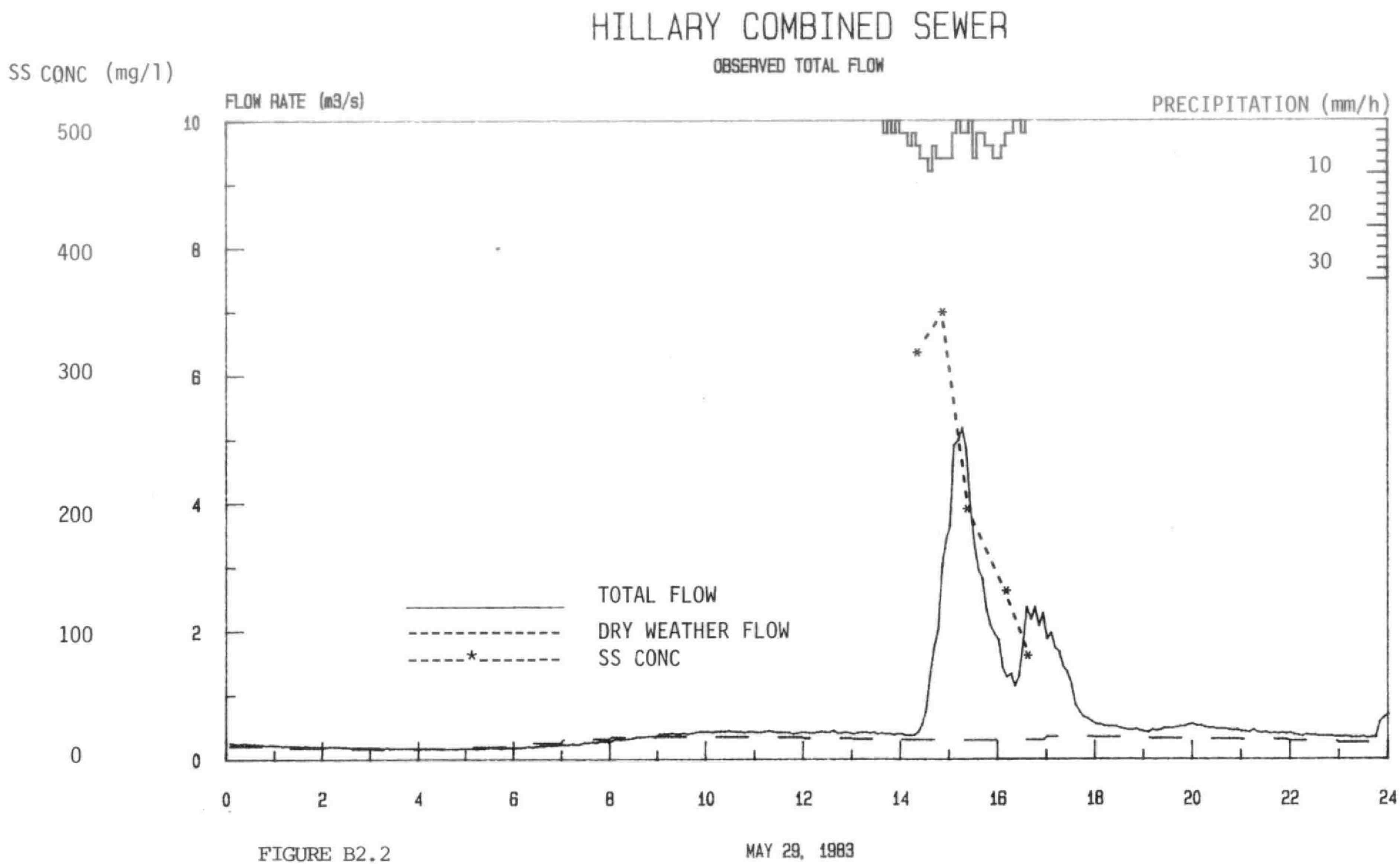



FIGURE B2.1

MAY 19, 1983



HILLARY COMBINED SEWER
OBSERVED TOTAL FLOW
SS CONC (mg/l)
FLOW RATE (m3/s)
PRECIPITATION (mm/h)
500
400
300
200
100
0
10
8
6
4
2
0
10
20
30
TOTAL FLOW
DRY WEATHER FLOW
SS CONC
0
2
4
6
8
10
12
14
16
18
20
22
24
MAY 29, 1983


FIGURE B2.2

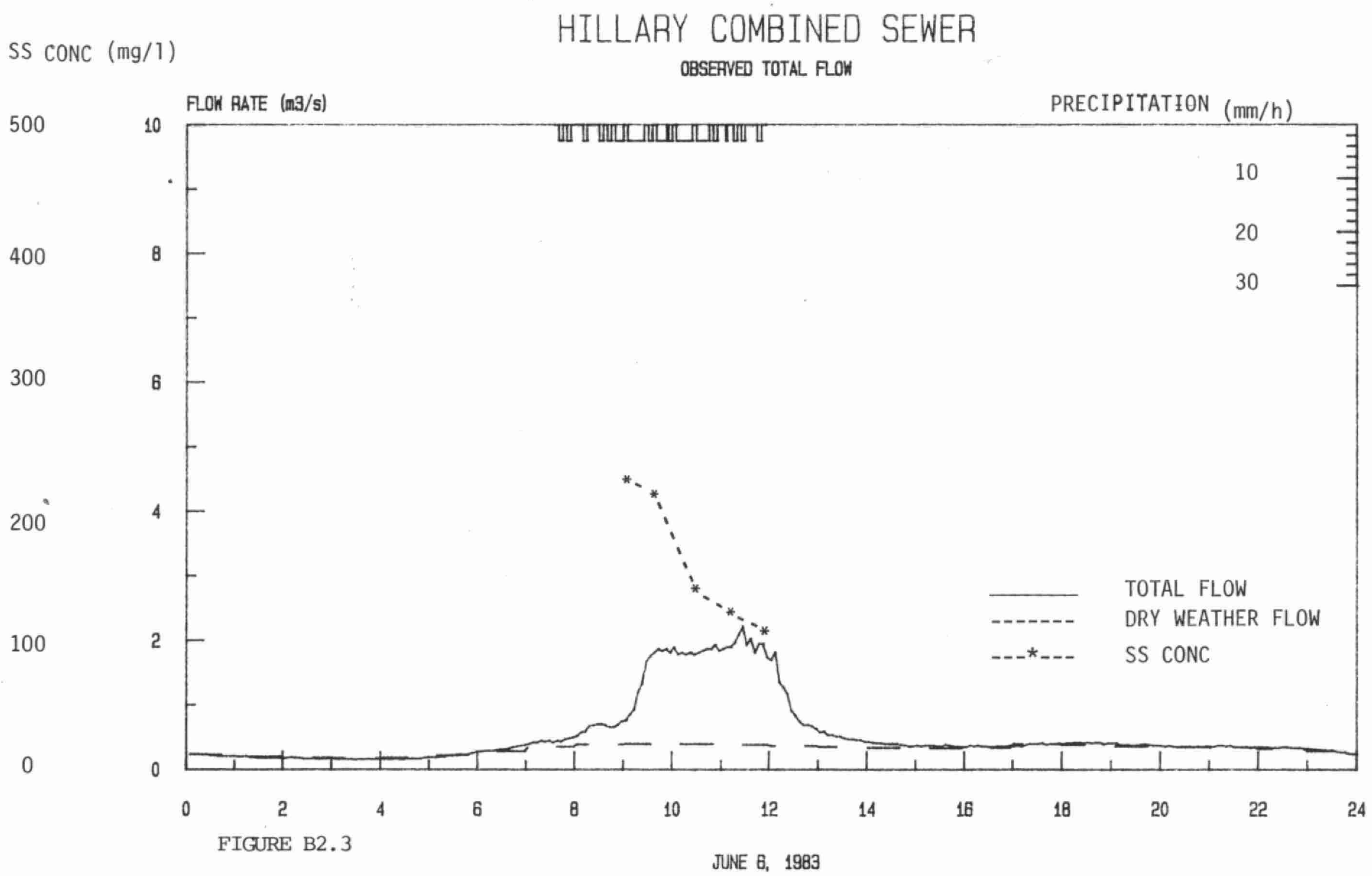

HILLARY COMBINED SEWER
OBSERVED TOTAL FLOW
SS CONC (mg/l)
FLOW RATE (m3/s)
PRECIPITATION (mm/h)
500
400
300
200
100
0
10
8
6
4
2
0
10
20
30
TOTAL FLOW
DRY WEATHER FLOW
SS CONC
0
2
4
6
8
10
12
14
16
18
20
22
24
JUNE 6, 1983

FIGURE B2.3

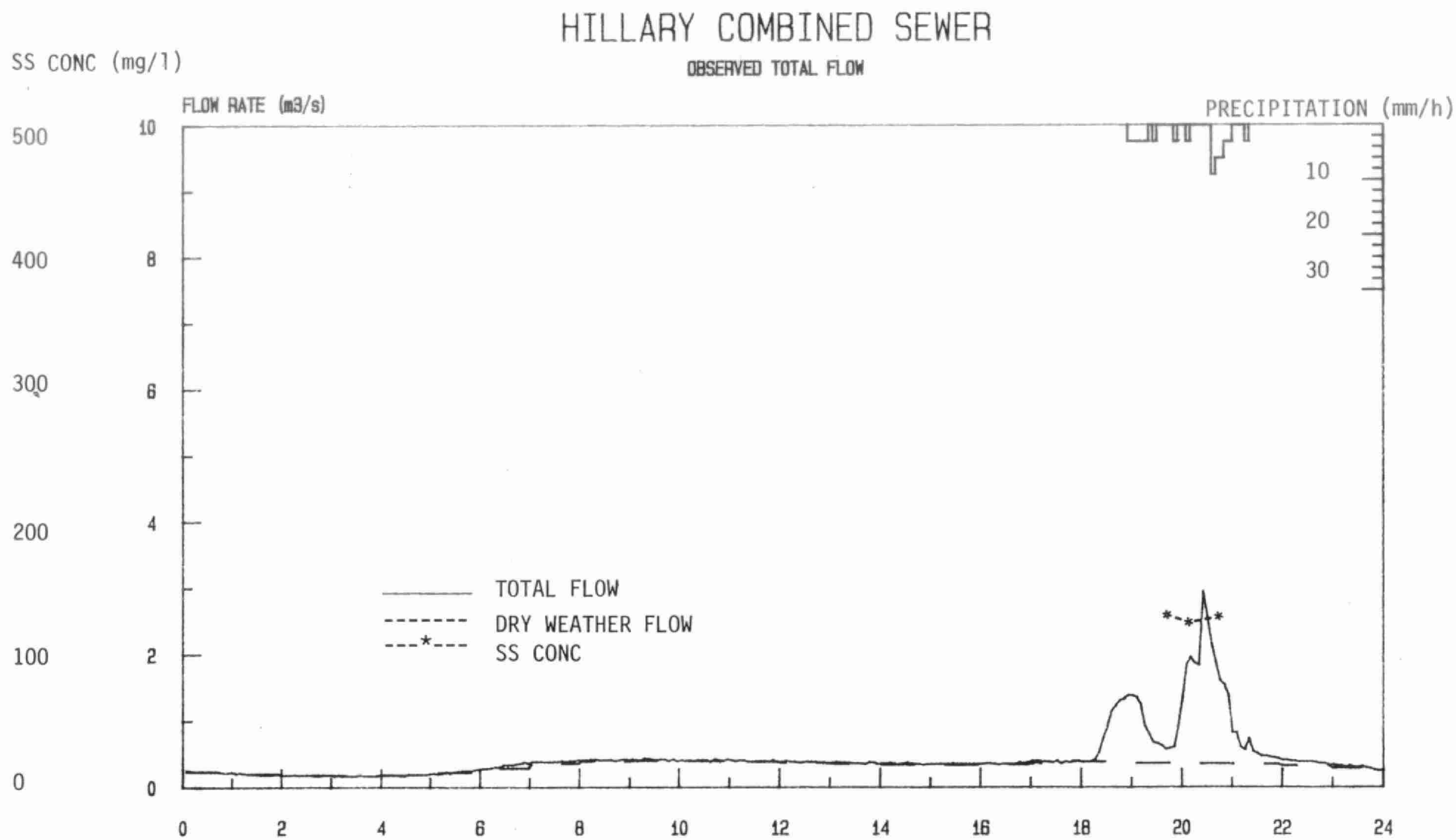
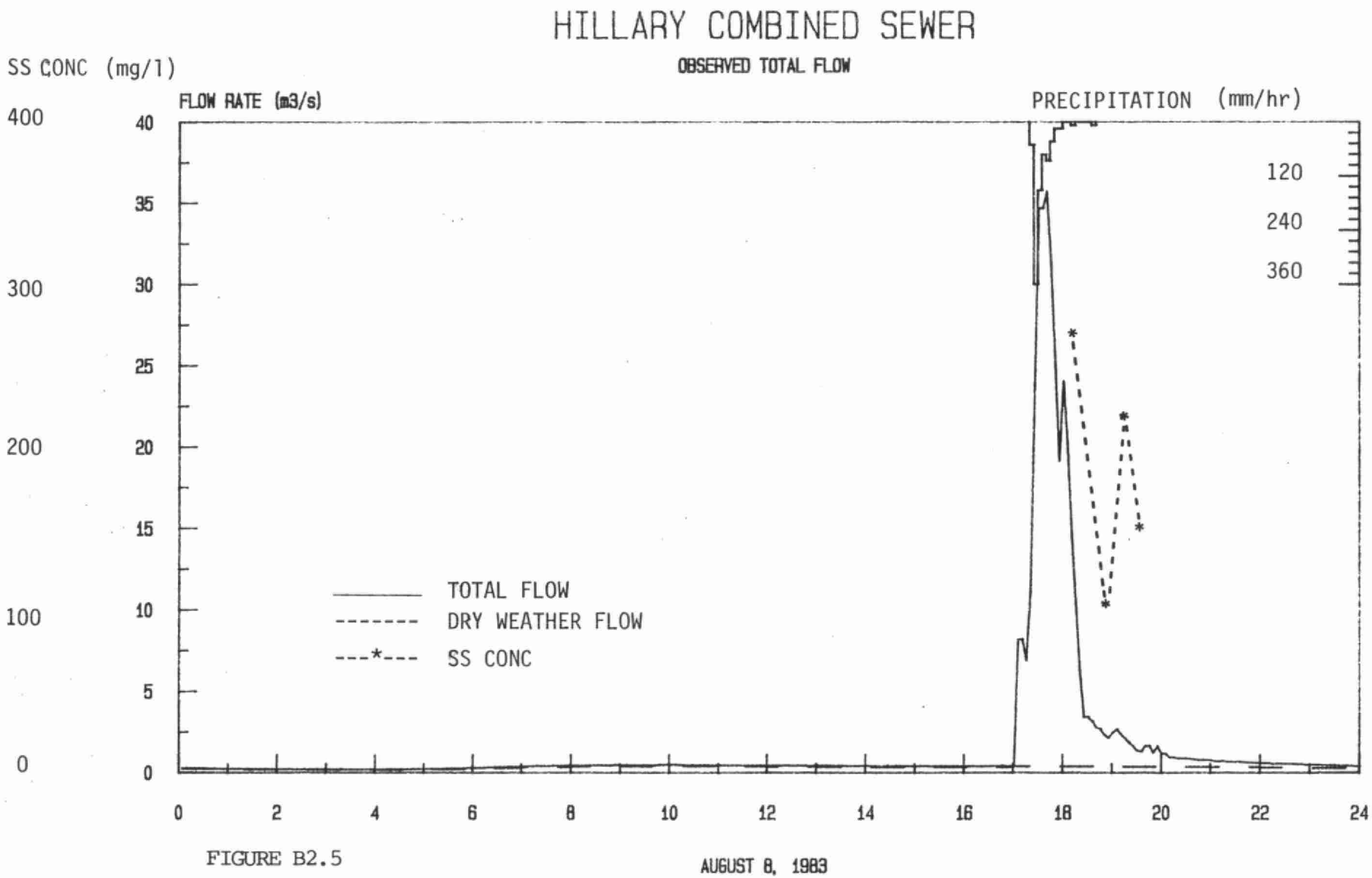



FIGURE B2.4

JULY 4, 1983



HILLARY COMBINED SEWER
OBSERVED TOTAL FLOW
SS CONC (mg/l)
FLOW RATE (m3/s)
PRECIPITATION (mm/hr)
400
300
200
100
0
40
35
30
25
20
15
10
5
0
120
240
360
TOTAL FLOW
DRY WEATHER FLOW
---*--- SS CONC
0
2
4
6
8
10
12
14
16
18
20
22
24
AUGUST 8, 1983


FIGURE B2.5

HILLARY COMBINED SEWER

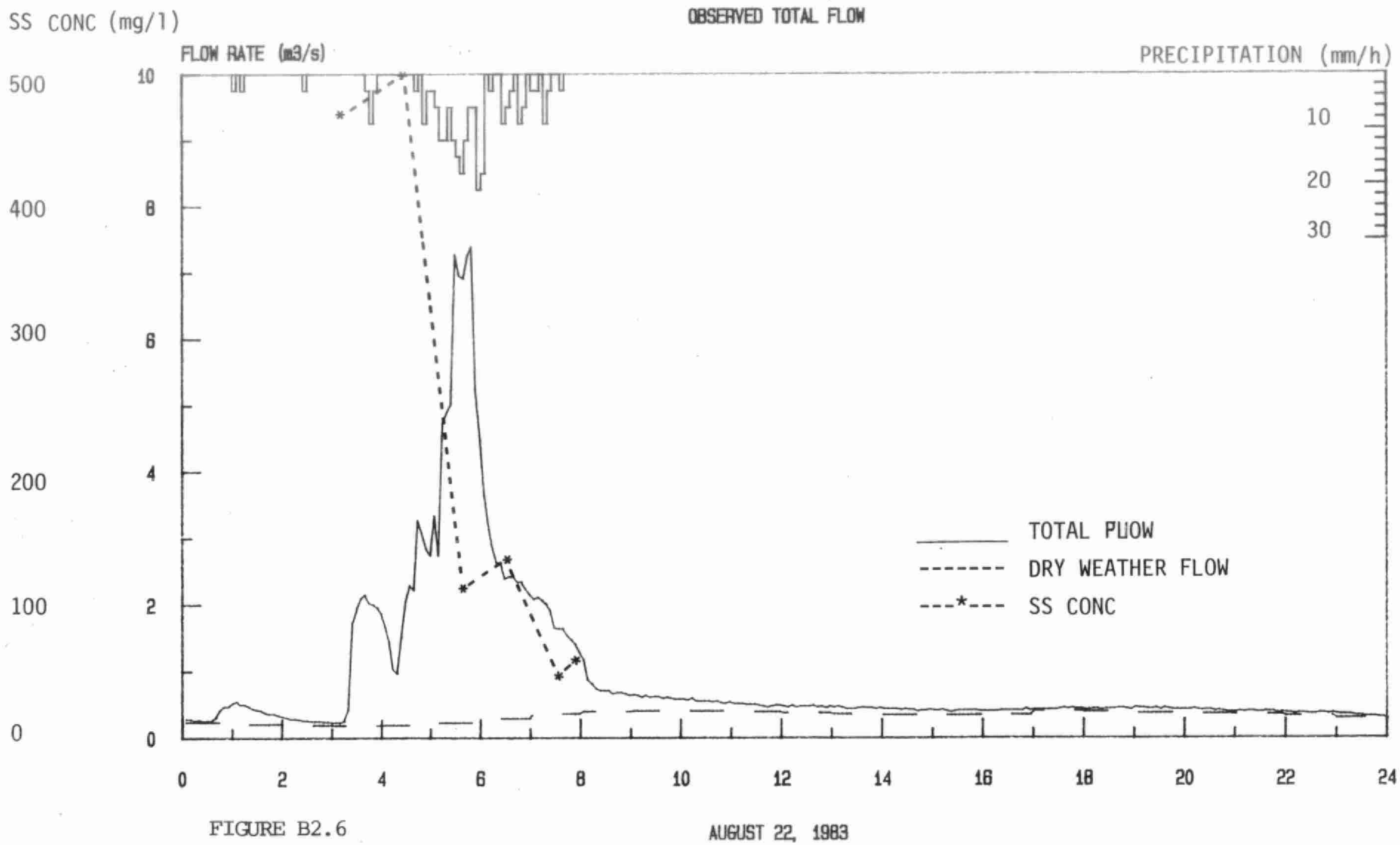



FIGURE B2.6

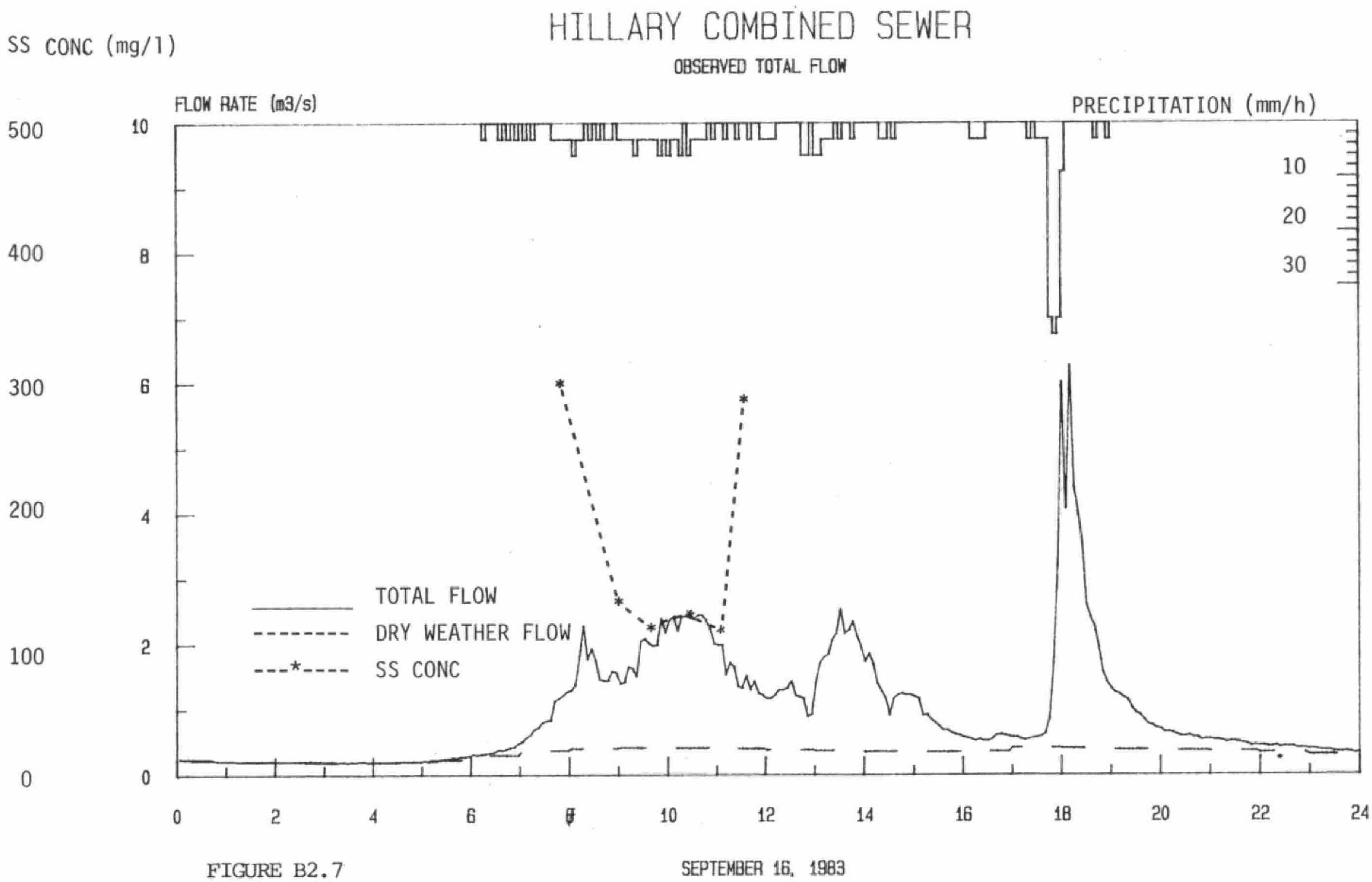

HILLARY COMBINED SEWER
OBSERVED TOTAL FLOW
SS CONC (mg/l)
FLOW RATE (m3/s)
PRECIPITATION (mm/h)
500
400
300
200
100
0
10
8
6
4
2
0
10
20
30
TOTAL FLOW
DRY WEATHER FLOW
---*---- SS CONC
0
2
4
6
8
10
12
14
16
18
20
22
24
SEPTEMBER 16, 1983

FIGURE B2.7

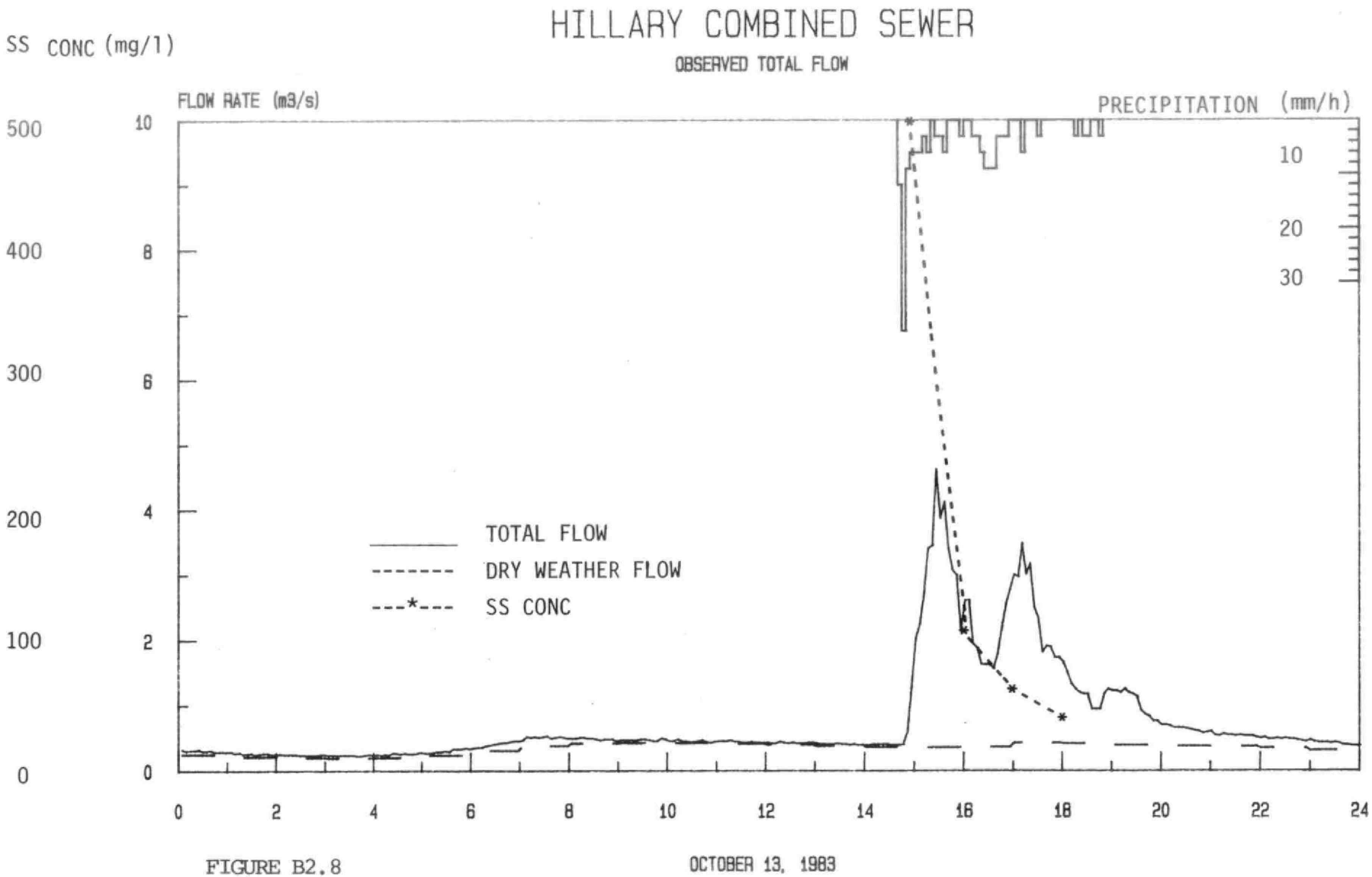

HILLARY COMBINED SEWER
OBSERVED TOTAL FLOW
SS CONC (mg/l)
FLOW RATE (m3/s)
PRECIPITATION (mm/h)
500
400
300
200
100
0
10
8
6
4
2
0
10
20
30
TOTAL FLOW
DRY WEATHER FLOW
---*--- SS CONC
0
2
4
6
8
10
12
14
16
18
20
22
24
OCTOBER 13, 1983

FIGURE B2.8

TABLE B2.4

OBSERVED COMBINED SEWAGE DATA (Sheet 1 of 5)

Sampling Station: 3 Event Date: 830519

| Sample Time | Flow Rate m3/s | PPUT mg/l | PPO4FR mg/l | RSP mg/l | BOD5 mg/l | Cd ug/l | Cu ug/l | Pb ug/l | Zn ug/l |
|---|---|---|---|---|---|---|---|---|---|
| 14:15 | 1.928 | 3.45 | 0.80 | 218.0 | 94.0 | 10 | 160 | 280 | 420 |
| 14:45 | 4.225 | 2.38 | 0.40 | 269.0 | 62.0 | 10 | 180 | 290 | 390 |
| 15:15 | 4.305 | 1.60 | 0.16 | 263.0 | 37.0 | 10 | 100 | 160 | 250 |
| 15:45 | 2.715 | 1.10 | 0.14 | 164.0 | 24.8 | 10 | 70 | 90 | 180 |
| 16:15 | 2.258 | 0.72 | 0.10 | 76.2 | 15.9 | 10 | 50 | 60 | 150 |
| 16:45 | 2.785 | 0.56 | 0.08 | 70.0 | 13.8 | 10 | 50 | 60 | 130 |
| 17:25 | 1.932 | 0.54 | 0.06 | 47.3 | 16.3 | 10 | 60 | 60 | 110 |
| 18:05 | 2.142 | 0.62 | 0.06 | 46.7 | 19.0 | 10 | 60 | 60 | 120 |

Sampling Station: 3 Event Date: 830529

| Sample Time | Flow Rate m3/s | PPUT mg/l | PPO4FR mg/l | RSP mg/l | BOD5 mg/l | Cd ug/l | Cu ug/l | Pb ug/l | Zn ug/l |
|---|---|---|---|---|---|---|---|---|---|
| 14:40 | 1.737 | 5.10 | 2.26 | 317.0 | 267.0 | | | | |
| 15:10 | 4.694 | 2.72 | 0.50 | 343.0 | 82.2 | | | | |
| 15:40 | 4.041 | 1.70 | 0.30 | 216.0 | 63.2 | | | | |
| 16:10 | 1.278 | 1.48 | 0.34 | 134.0 | 45.4 | | | | |
| 16:40 | 1.992 | 1.35 | 0.38 | 92.2 | 36.8 | | | | |

Notes :
PPUT = Total Phosphorous RSP = Residue Particulate.
PPO4FR = Filtered Reactive Phosphorous

TABLE B2.4

OBSERVED COMBINED SEWAGE DATA (Sheet 2 of 5)

Samlplng Station: 3 Event Date: 830606

| Sample Time | Flow Rate m3/s | PPUT mg/l | PPO4FR mg/l | RSP mg/l | BOD5 mg/l | Cd ug/l | Cu ug/l | Pb ug/l | Zn ug/l |
|---|---|---|---|---|---|---|---|---|---|
| 09:15 | 1.199 | 3.75 | 1.64 | 214 | 103.0 | | | | |
| 10:00 | 1.897 | 2.75 | 1.08 | 201 | 70.3 | | | | |
| 10:45 | 1.868 | 1.65 | 0.50 | 132 | 38.6 | | | | |
| 11:30 | 1.928 | 1.33 | 0.38 | 114 | 21.6 | | | | |
| 12:00 | 1.694 | 1.38 | 0.36 | 104 | 21.4 | | | | |

Sampling Station: 3 Event Date: 830704

| Sample Time | Flow Rate m3/s | PPUT mg/l | PPO4FR mg/l | RSP mg/l | BOD5 mg/l | Cd ug/l | Cu ug/l | Pb ug/l | Zn ug/l |
|---|---|---|---|---|---|---|---|---|---|
| 20:00 | 1.331 | 1.70 | 0.28 | 135.0 | 51.0 | 5 | 120 | 170 | 300 |
| 20:15 | 1.868 | 1.75 | 0.30 | 126.0 | 58.0 | 5 | 90 | 170 | 260 |
| 20:30 | 2.024 | 1.85 | 0.30 | 135.0 | 55.0 | 5 | 120 | 180 | 280 |
| 20:45 | 1.637 | 1.85 | 0.28 | 131.0 | 58.0 | 5 | 340 | 180 | 340 |

Notes :
PPUT = Total Phosphorous
PPO4FR = Filtered Reactive Phosphorous
RSP = Residue Particulate.

TABLE B2.4

OBSERVED COMBINED SEWAGE DATA (Sheet 3 of 5)

Samlplng Station: 3 Event Date: 830808

| Sample Time | Flow Rate m3/s | PPUT mg/l | PPO4FR mg/l | RSP mg/l | BOD5 mg/l | Cd ug/l | Cu ug/l | Pb ug/l | Zn ug/l |
|---|---|---|---|---|---|---|---|---|---|
| 17:20 | 2.3563 | 3.70 | 1.60 | 1120.0 | 145.0 | 5 | 150 | 70 | 160 |
| 17:50 | 6.3143 | 2.75 | 0.10 | 429.0 | 45.6 | 5 | 160 | 640 | 910 |
| 18:20 | 7.012 | 2.30 | 0.06 | 271.0 | 43.2 | 5 | 130 | 500 | 710 |
| 18:50 | 2.116 | 1.75 | 0.06 | 103.0 | 34.8 | 5 | 130 | 410 | 570 |
| 19:20 | 1.855 | 2.13 | 0.04 | 214.0 | 41.5 | 5 | 130 | 430 | 820 |
| 19:50 | 1.224 | 1.30 | 0.02 | 150.0 | 42.1 | 5 | 110 | 380 | 610 |

Sampling Station: 3 Event Date: 830811

| Sample Time | Flow Rate m3/s | PPUT mg/l | PPO4FR mg/l | RSP mg/l | BOD5 mg/l | Cd ug/l | Cu ug/l | Pb ug/l | Zn ug/l |
|---|---|---|---|---|---|---|---|---|---|
| 06:00 | NA | 2.50 | 0.98 | 135.0 | | 4 | 120 | 110 | 230 |
| 06:15 | NA | 2.00 | 0.68 | 179.0 | | 4 | 100 | 150 | 240 |
| 06:30 | NA | 2.05 | 0.68 | 135.0 | | 4 | 140 | 140 | 200 |
| 06:45 | NA | 2.05 | 0.52 | 133.0 | | 4 | 110 | 110 | 170 |
| 07:00 | NA | 2.00 | 0.56 | 84.1 | | 4 | 110 | 90 | 170 |
| 07:15 | NA | 2.03 | 0.58 | 76.1 | | 5 | 70 | 130 | 170 |

Notes :
PPUT = Total Phosphorous RSP = Residue Particulate.
PPO4FR = Filtered Reactive Phosphorous
NA = Not Available, Flow Recorder Malfunction.

TABLE B2.4

OBSERVED COMBINED SEWAGE DATA (Sheet 4 of 5)

Samlplng Station: 3 Event Date: 830822

| Sample Time | Flow Rate m3/s | PPUT mg/l | PPO4FR mg/l | RSP mg/l | BOD5 mg/l | Cd ug/l | Cu ug/l | Pb ug/l | Zn ug/l |
|---|---|---|---|---|---|---|---|---|---|
| 03:25 | 2.598 | 3.2 | 0.56 | 465.0 | 148.0 | 4 | 310 | 530 | 720 |
| 04:25 | 2.872 | 5.03 | 0.80 | 609.0 | 173.0 | 5 | 180 | 420 | 740 |
| 05:25 | 3.990 | | 0.12 | 110.0 | 25.8 | 4 | 70 | 110 | 180 |
| 06:25 | 2.167 | | 0.10 | 124.0 | 13.4 | 4 | 50 | 70 | 150 |
| 07:25 | 1.642 | | 0.18 | 44.1 | 13.6 | 4 | 40 | 50 | 160 |
| 07:50 | 1.403 | | 0.44 | 59.2 | 18.6 | 4 | 50 | 40 | 140 |

Sampling Station: 3 Event Date: 830916

| Sample Time | Flow Rate m3/s | PPUT mg/l | PPO4FR mg/l | RSP mg/l | BOD5 mg/l | Cd ug/l | Cu ug/l | Pb ug/l | Zn ug/l |
|---|---|---|---|---|---|---|---|---|---|
| 08:10 | 1.674 | 4.46 | 1.24 | 294.0 | 111.0 | 6 | 240 | 220 | 450 |
| 08:55 | 1.606 | 2.02 | 0.56 | 126.0 | 41.0 | 6 | 170 | 130 | 290 |
| 09:40 | 1.830 | 1.52 | 0.46 | 106.0 | 40.0 | 6 | 150 | 80 | 240 |
| 10:30 | 1.857 | 1.30 | 0.36 | 114.0 | 38.0 | 6 | 160 | 160 | 270 |
| 11:10 | 1.758 | 2.50 | 0.32 | 106.0 | 32.0 | 6 | 120 | 90 | 220 |
| 11:50 | 1.476 | 4.75 | 1.40 | 288.0 | 120.0 | 6 | 350 | 220 | 470 |

Notes :
PPUT = Total Phosphorous
RSP = Residue Particulate
PPO4FR = Filtered Reactive Phosphorous

TABLE B2.4

OBSERVED COMBINED SEWAGE DATA (Sheet 5 of 5)

Samlplng Station: 3 Event Date: 831012

| Sample Time | Flow Rate m3/s | PPUT mg/l | PPO4FR mg/l | RSP mg/l | BOD5 mg/l | Cd ug/l | Cu ug/l | Pb ug/l | Zn ug/l |
|---|---|---|---|---|---|---|---|---|---|
| 05:15 | 1.760 | 4.03 | 1.04 | | 142.0 | 6 | 160 | 120 | 260 |
| 05:45 | 1.785 | 4.10 | 1.06 | | 150.0 | 6 | 180 | 160 | 280 |
| 06:15 | 1.674 | 1.55 | 0.32 | | 72.0 | 6 | 130 | 150 | 200 |
| 06:45 | 1.642 | 2.15 | 0.22 | | 38.0 | 6 | 110 | 160 | 210 |
| 08:40 | 1.714 | 1.00 | 0.22 | | 28.0 | 5 | 60 | 90 | 110 |
| 09:00 | 1.356 | 0.65 | 0.18 | | 27.0 | 5 | 70 | 90 | 130 |

Sampling Station: 3 Event Date: 831013

| Sample Time | Flow Rate m3/s | PPUT mg/l | PPO4FR mg/l | RSP mg/l | BOD5 mg/l | Cd ug/l | Cu ug/l | Pb ug/l | Zn ug/l |
|---|---|---|---|---|---|---|---|---|---|
| 15:00 | 2.901 | 3.40 | 0.54 | 525.0 | 95.0 | 5 | 160 | 230 | 390 |
| 16:00 | 2.506 | 0.84 | 0.16 | 105.0 | 54.0 | 5 | 60 | 130 | 180 |
| 17:00 | 2.901 | 0.89 | 0.38 | 71.6 | 23.0 | 5 | 50 | 100 | 140 |
| 18:00 | 1.447 | 0.80 | 0.26 | 50.3 | 48.0 | 5 | 50 | 80 | 120 |
| | | | | | | 5 | | | |
| | | | | | | 5 | | | |

Notes :
PPUT = Total Phosphorous
PPO4FR = Filtered Reactive Phosphorous
RSP = Residue Particulate

TABLE B2.5

BACTERIAL CONCENTRATIONS IN COMBINED SEWAGE (Sheet 1 of 2)

| Event | Site | Date | F.C. Counts,100ml. | Log | F.S. Counts,10ml. | Log |
|---|---|---|---|---|---|---|
| 6452 | 8 | 830830 | 980000 | 5.591 | 870000 | 5.940 |
| | 8 | | 9400000 | 6.973 | 730000 | 5.863 |
| | 8 | | 9600000 | 6.982 | 760000 | 5.881 |
| | 8 | | 530000 | 5.724 | 420000 | 5.623 |
| | 8 | | 1000 | 3.000 | 230000 | 5.362 |
| | 9 | | 470000 | 6.672 | 190000 | 5.279 |
| | 9 | | 480000 | 6.681 | 75000 | 4.875 |
| | 9 | | 460000 | 6.663 | 210000 | 5.322 |
| | 3 | | 830000 | 6.919 | 910000 | 5.959 |
| | 8 | | 530000 | 6.724 | 1900000 | 6.279 |
| | 9 | | 3100000 | 6.491 | 390000 | 5.591 |
| 5377 | 3 | 830808 | 9400000 | 6.973 | 75000 | 4.875 |
| | 3 | | 670000 | 5.826 | 31000 | 4.491 |
| | 3 | | 690000 | 5.839 | 41000 | 4.613 |
| | 3 | | 430000 | 5.633 | 39000 | 4.591 |
| | 3 | | 500000 | 5.699 | 35000 | 4.544 |
| | 3 | | 450000 | 5.653 | 37000 | 4.568 |
| 5375 | 3 | 830804 | 3300000 | 6.519 | 380000 | 5.580 |
| | 3 | | 2300000 | 6.362 | 380000 | 5.580 |
| | 3 | | 2000000 | 6.301 | 290000 | 5.462 |
| | 3 | | 3000000 | 6.477 | 350000 | 5.544 |
| 5370 | 3 | 830704 | 3400000 | 6.531 | 240000 | 5.380 |
| | 3 | | 3900000 | 6.591 | 360000 | 5.556 |
| | 3 | | 3900000 | 6.591 | 370000 | 5.568 |
| | 3 | | 4400000 | 6.643 | 310000 | 5.491 |
| | 8 | | 1.33E+07 | 7.124 | 1180000 | 6.072 |
| | 8 | | 1.43E+07 | 7.155 | 650000 | 5.813 |
| | 8 | | 1.36E+07 | 7.134 | 680000 | 5.833 |
| | 8 | | 8900000 | 6.949 | 1080000 | 6.033 |
| | 8 | | 9600000 | 6.982 | 1430000 | 6.155 |
| | 8 | | 1.26E+07 | 7.100 | 1490000 | 6.173 |
| | 9 | | 8400000 | 6.924 | 1250000 | 6.097 |
| | 9 | | 8300000 | 6.919 | 180000 | 5.255 |
| | 9 | | 9200000 | 6.964 | 240000 | 5.380 |
| | 9 | | 5900000 | 6.771 | 260000 | 5.415 |
| 5367 | 3 | 830606 | 8700000 | 5.940 | 130000 | 5.114 |
| | 8 | | 4700000 | 5.672 | 150000 | 5.176 |
| | 9 | | 9700000 | 5.987 | 170000 | 5.230 |
| 5366 | 3 | 830529 | 8300000 | 6.919 | 770000 | 5.886 |
| | | | 1900000 | 6.279 | 420000 | 5.623 |
| | | | 1060000 | 6.025 | 250000 | 5.398 |
| | | | 9100000 | 5.959 | 150000 | 5.176 |
| | | | 1600000 | 6.204 | 113000 | 5.053 |
| | | | 1270000 | 6.104 | 83000 | 4.919 |

TABLE B2.5

BACTERIAL CONCENTRATIONS IN COMBINED SEWAGE (Sheet 2 of 2)

| Event | Site | Date | F.C. Counts,100ml. | Log | F.S. Counts,10ml. | Log |
|---|---|---|---|---|---|---|
| 5394 | 3 | 830811 | 370000 | 5.568 | 280000 | 5.447 |
| | 3 | | 190000 | 5.279 | 72000 | 4.857 |
| | 3 | | 180000 | 5.255 | 95000 | 4.978 |
| | 3 | | 96000 | 4.982 | 52000 | 4.716 |
| | 3 | | 460000 | 5.663 | 64000 | 4.806 |
| | 3 | | 480000 | 5.681 | 47000 | 4.672 |
| | 3 | | 97000 | 4.987 | 94000 | 4.973 |
| | 8 | | 240000 | 5.380 | 2200000 | 6.342 |
| | 8 | | 22000 | 4.342 | 870000 | 3.940 |
| | 9 | | 32000 | 4.505 | 72000 | 4.857 |
| | 9 | | 360000 | 5.556 | 46000 | 4.663 |
| 5393 | 3 | 830811 | 5300000 | 6.724 | 650000 | 5.813 |
| | 3 | | 5500000 | 6.740 | 460000 | 5.663 |
| | 3 | | 4200000 | 6.623 | 280000 | 5.447 |
| | 3 | | 3300000 | 6.519 | 290000 | 5.462 |
| | 3 | | 2900000 | 6.462 | 270000 | 5.431 |
| | 3 | | 2700000 | 6.431 | 230000 | 5.362 |
| | 8 | | 7100000 | 6.851 | 360000 | 5.556 |
| | 8 | | 4300000 | 6.633 | 320000 | 5.505 |
| | 8 | | 3900000 | 6.591 | 220000 | 5.342 |
| | 8 | | 2800000 | 6.447 | 290000 | 5.462 |
| | 8 | | 2900000 | 6.462 | 310000 | 5.491 |
| 6467 | 3 | 831115 | 1050000 | 6.021 | 220000 | 5.342 |
| | | | 850000 | 5.929 | 140000 | 5.146 |
| | | | 710000 | 5.851 | 110000 | 5.041 |
| 6466 | 3 | 831116 | 1700000 | 6.230 | 100000 | 5.000 |
| 6463 | 3 | 831115 | 990000 | 5.996 | 5500000 | 6.740 |
| 6457 | 3 | 831013 | 3400000 | 6.531 | 360000 | 5.556 |
| | 3 | | 1080000 | 6.033 | 150000 | 5.176 |
| | 3 | | 1240000 | 6.093 | 320000 | 5.505 |
| | 3 | | 2300000 | 6.362 | 530000 | 5.724 |
| 6455 | 3 | 381012 | 6100000 | 6.785 | 730000 | 5.868 |
| | 3 | | 5700000 | 6.756 | 940000 | 5.973 |
| | 3 | | 1190000 | 6.076 | 300000 | 5.477 |
| | 3 | | 930000 | 5.968 | 270000 | 5.431 |
| | 3 | | 580000 | 5.763 | 220000 | 5.342 |
| | 3 | | 970000 | 5.987 | 230000 | 5.362 |
| Log Mean | | | | 6.218 | | 5.421 |
| Std. Dev. | | | | 0.700 | | 0.462 |

# APPENDIX B3

# INFLOW / INFILTRATION

## WET WEATHER I/I FROM SANITARY SEWER AREA

### Method of Calculation

(1) Daily volumes of dry days were extracted from WPCP records

(a) there had to be at least 2 days pass after a precipitation event.

(2) The average daily dry weather flow for individual months and each year were calculated.

(3) The wet weather days were identified

(a) any day with precipitation
(b) plus the day after the precipitation.

(4) The net extraneous flow was calculated

(a) the average monthly dry weather flow was subtracted from the individual wet flow record for each day between April and October.

(5) The wet days were selected for the regression table

(a) precipitation had to be greater than 5.0 mm

(b) the net extraneous flow for the day of precipitation had to have a positive value

(c) the net extraneous flow for the day after the precipitation event had to have a positive value or, if negative, the sum of the two days had to be positive

(d) if two consecutive events greater than 5.0 mm occurred then

1) the two days of precipitation were added together

2) the two days of flow plus the day after were added together.

(6) Use equation of the form:

$$II = A * P^B,$$

where II = Event I/I in 1,000 $m^3$

P = Event precipitation in mm

A and B are constants

(7) Obtain regression results for the April-October season for each year from 1980 through 1983. Select the year with best coefficient of correlation.

(8) Apportion II volume to each hour of event according to precipitation distribution. Synthesize hydrograph for the distributed II, using a unit hydrograph of the WPCP.

TABLE B3.1

WET WEATHER I/I AT HUMBER WPCP (1)

| Month (1980) | Avg DWF for month ($10^3m^3$) | Event I/I Volume ($10^3m^3$) | Precipitation (mm) |
|---|---|---|---|
| April | 350.9 | 89.2 | 15.2 |
| | | 434.3 | 29.4 |
| | | 242.4 | 20.1 |
| May | 333.6 | 61.3 | 12.8 |
| | | 39.3 | 20.5 |
| June | 408.8 | 42.6 | 20.5 |
| July | 359.0 | 124.8 | 7.3 |
| | | 341.7 | 25.2 |
| | | 672.9 | 62.1 |
| August | 391.1 | 172.0 | 5.8 |
| September | 427.1 | 16.3 | 9.1 |
| | | 90.0 | 6.2 |
| | | 85.9 | 14.4 |
| | | 1.7 | 5.1 |
| October | 456.4 | 51.2 | 18.8 |
| | | 2.9 | 5.0 |
| | | 124.1 | 35.4 |

$I/I = 1.475 * P^{1.446}$

Where I/I = Event inflow/infiltration, $10^3m^3$

P = Event precip. mm.

Coeff. of correl. = 0.663

Note:

(1) For events greater than 5.0 mm.

**APPENDIX C1**

**MODEL INPUT**

TABLE C 1.1 (Sheet 1 of 6)

```
  10   8
   1   2   3   4
RUNOFF
A1   *** TAWMS COMBINED SEWER ANALYSIS *** HUMBER SEWERSHED
A1   1983 CATCHMENTS
B1   0 1  0    1    0    1    1   16    0   11   07   79
B2        40  15.  90. 0.01
E1         4 60.0
E2       7.8 28.1   .0   .2
F1       0.0  0.0  1.3  3.3  3.3  4.1  7.0  4.1  2.2  0.8  0.1  0.0
G1       331  337    3
G1       332  330    3
G1       333  330    3
G1       334  330    3
G1       335  330    3
G1       336  337    3
G1       337  330    3
G1       341  347    3
G1       342  340    3
G1       343  340    3
G1       344  340    3
G1       345  340    3
G1       346  347    3
G1       347  340    3
G1       351  357    3
G1       352  350    3
G1       353  350    3
G1       354  350    3
G1       355  350    3
G1       356  357    3
G1       357  350    3
G1       401  410    3
G1       402  410    3
G1       403  410    3
G1       404  410    3
G1       405  410    3
G1       501  510    3
G1       504  510    3
G1       505  510    3
G2
H1        -1        1.0
H1        -2                             .0150  .30 -2.0 10.0  7.3 0.50 .00025
H1   1 2531   3311351.135.1 18.9   .06
H1   1 2532   332    9.   0.9 50.0   .06
H1   1 2533   333 184. 18.4100.0   .06
H1   1 2534   334 217. 21.7 42.0   .06
H1   1 2535   335 222. 22.2  5.0   .06
H1   1 2536   336 321. 32.1 18.9   .06              7.00
H1   1 2541   3413018.301.8 18.9   .06
H1   1 2542   342   20.   2.0 50.0   .06
H1   1 2543   343 411. 41.1100.0   .06
H1   1 2544   344 485. 48.5 42.0   .06
H1   1 2545   345 496. 49.6  5.0   .06
H1   1 2546   346 717. 71.7 18.9   .06              7.00
H1   1 2551   351 808. 80.8 18.9   .06
H1   1 2552   352    6.    .6 50.0   .06
H1   1 2553   353 110. 11.0100.0   .06
H1   1 2554   354 130. 13.0 42.0   .06
H1   1 2555   355 133. 13.3  5.0   .06
```

```
H1   1 2556   356 192. 19.2 18.9   .06                         7.00
H1   1 2151   4011489.101.5 33.0   .02
H1   1 2152   402   59.   4.0 50.0   .02
H1   1 2153   403 213. 14.5 100.   .02
H1   1 2154   404 499. 34.0 90.0   .02
H1   1 2155   405 191. 13.0 10.0   .02
H1   1 2181   5011452.149.8 28.0 .015
H1   1 2184   504 237. 24.5 95.0 .015
H1   1 2185   505 211. 21.8   2.0 .015
H2
J1   8    5    0    0  13.  0.0            0.30
J2            -2           200.      0.40        26.7  7.0        7.0
J2  FAMILY     0    0                                        .189
J2 MFAMILY     0    0                                        .500
J2COMMERCE     0    0                                        1.00
J2INDUSTRY     0    0                                        .420
J2  OTHERS     0    0                                        .050
J3                    -2                                                     .3
J3 S.S.     MG/L    0   0 2      1000.1000.1000.1000.1000.1.6622.138
J3 BOD5     MG/L    0   0 2       200. 200. 200. 200. 200.1.5001.164
J3 SOLU-P.  MG/L    0   4 1                                1.0 .432
J3 TOTAL-P  MG/L    0   0 2        9.2  9.2  9.2  9.2  9.21.662.0197
J3 CADMIUM  MG/L    0   0 2       .038 .038 .038 .038 .0381.6628.E-5
J3 COPPER   MG/L    0   0 2       .645 .645 .645 .645 .6451.6621.E-3
J3 LEAD     MG/L    0   0 2       1.13 1.13 1.13 1.13 1.131.6622.E-3
J3 ZINC     MG/L    0   0 2       1.89 1.89 1.89 1.89 1.891.6624.E-3
J5
L1      2531    1           19.8
L1      2532    2            0.5
L1      2533    3           14.6
L1      2534    4            7.3
L1      2535    5            1.0
L1      2536    1            4.7
L1      2541    1           44.3
L1      2542    2            1.2
L1      2543    3           32.6
L1      2544    4           16.3
L1      2545    5            2.3
L1      2546    1           10.5
L1      2551    1           11.8
L1      2552    2            0.3
L1      2553    3            8.7
L1      2554    4            4.4
L1      2555    5            0.6
L1      2556    1            2.8
L1      2151    1           14.2
L1      2152    2            0.8
L1      2153    3            6.2
L1      2154    4           13.0
L1      2155    5            0.6
L1      2181    1           20.6
L1      2184    4           11.4
L1      2185    5            0.2
M1   5    1
M2330  340  350  410  510
ENDPROGRAM
```

```
   8   9
   1   2
TRANSPORT
A1
C1   *** TAWMS.COMBINED SEWER ANALYSIS *** HUMBER SEWERSHED
C1     1983 REGULATORS.
D1       40     4     0   13     0     1    4     4     790711    1    1
D2      900.        .001       13.      16.00        .01
D3   0      0     0     0
E1 330                   16 0.08140   0.014    0.160     0.06     0.22
E1 335 330               18              .42                                      336.0
E1 336 335               16
E1 340                   16  0.1822   0.014    0.160     0.06     0.22
E1 345 336 340           16
E1 350                   16 0.04860   0.014    0.160     0.06     0.22
E1 355 350               18            0.27                                       356.0
E1 356 355               16
E1 360 345               16
E1 362 360               18            2.75                                       363.0
E1 363 362 356           16
E1 364 363               18            1.64                                       365.0
E1 365 364               16
E1 410                   16 0.06300   0.014    0.160     0.06     0.22
E1 420 410               18           0.324                                       422.0
E1 422 420               16
E1 510                   16 0.05000   0.014    0.160     0.06     0.22
E1 520 510               18           0.474                                       522.0
E1 522 520               16
E1  50                   16    .6883  0.014    0.370     0.04     0.17
E1 850  50                1    1720.  1.634    .1663    .0209              1.0
E1  42 850               16    .3846  0.014    0.370     0.04     0.17
E1 842  42                1    3950.  1.727    .2846    .0209              1.0
E1  24 842               16
E1 824  24                1     930.  1.524    .5054    .0209              1.0
E1  21 824               16     .368  0.014    0.370     0.04     0.17
E1 821  21                1    1750.  1.667    .5600    .0209              1.0
E1  14 821               16     .104  0.014    0.370     0.04     0.17
E1 814  14                1     870.  1.829    .2529    .0209              1.0
E1  11 814 860           16
E1  10  11 522           16    .0129  0.014    0.370     0.04     0.17
E1 810  10                1    2310.  2.800    .2251    .0209              1.0
E1   6 810               16     .525  0.014    0.370     0.04     0.17
E1 806   6                1    1980.  3.500    .1414    .0209              1.0
E1   2 806               16     .116  0.014    0.370     0.04     0.17
E1 770   2               20    10.15   2.27     38.1    26.22    3.189               1.
E1 802   1                1     350.  3.500    .2097    .0209              1.0
E1   1 770               16
E1  94                   16     .644  0.014    0.370     0.04     0.17
E1  96                   16    .0388  0.014    0.370     0.04     0.17
E1  98 802   94   96     16
E1 772  98               16
E1 774 772               18           11.73                                       776.
E1 776 774               16
E1 150                   16     1.00  0.014    0.370     0.04     0.17
E1 155 365 150 220       16
E1 220                   16    .0171
E1 160 155 422           16
E1 860 160               16
E1  99 776               16
```

TABLE C 1.1 (Sheet 4 of 6)

```
E2
F1   5CADMIUM    MG/L     0
F1   6COPPER     MG/L     0
F1   7LEAD       MG/L     0
F1   8ZINC       MG/L     0
I1 24   160     6  772
J2330   340   350  363  365  410  422  510  522    2    1  772  776
R1      16.0       0.0
R1      16.0       0.0
R1      16.0       0.0
R1      16.0       0.0
R1      20.5       4.44
R1      20.5       3.79
R1      20.5       4.61
R1      20.5       3.62
R1      26.0       0.0
R1      26.0       0.0
R1      26.0       0.0
R1      26.0       0.0
ENDPROGRAM
```

TABLE C 1.1 (Sheet 5 Of 6)

```
   8   9
   1   2
TRANSPORT
A1
C1  *** TAWMS.COMBINED SEWER ANALYSIS *** HUMBER SEWERSHED
C1       1983 REGULATORS.
D1       40    4    0   13    0    1    4    4    790711    1    1
D2     900.      .001       13.     16.00        .01
D3  0     0     0     0
E1 330                 16 0.08140   245.4   208.9    2.42    5.42
E1 335 330             18             .42                                      336.0
E1 336 335             16
E1 340                 16  0.1822   245.4   208.9    2.42    5.42
E1 345 336 340         16
E1 350                 16 0.04860   245.4   208.9    2.42    5.42
E1 355 350             18            0.27                                      356.0
E1 356 355             16
E1 360 345             16
E1 362 360             18            2.75                                      363.0
E1 363 362 356         16
E1 364 363             18            1.64                                      365.0
E1 365 364             16
E1 410                 16 0.06300   245.4   208.9    2.42    5.42
E1 420 410             18           0.324                                      422.0
E1 422 420             16
E1 510                 16 0.05000   245.4   208.9    2.42    5.42
E1 520 510             18           0.474                                      522.0
E1 522 520             16
E1  50                 16   .6883   173.8   191.0    2.58    5.66
E1 850  50              1   1720.   1.634   .1663   .0209           1.0
E1  42 850             16   .3846   173.8   191.9    2.58    5.66
E1 842  42              1   3950.   1.727   .2846   .0209           1.0
E1  24 842             16
E1 824  24              1    930.   1.524   .5054   .0209           1.0
E1  21 824             16    .368   173.8   191.9    2.58    5.66
E1 821  21              1   1750.   1.667   .5600   .0209           1.0
E1  14 821             16    .104   173.8   191.9    2.58    5.66
E1 814  14              1    870.   1.829   .2529   .0209           1.0
E1  11 814 860         16
E1  10  11 522         16   .0129   173.8   191.9    2.58    5.66
E1 810  10              1   2310.   2.800   .2251   .0209           1.0
E1   6 810             16    .525   173.8   191.9    2.58    5.66
E1 806   6              1   1980.   3.500   .1414   .0209           1.0
E1   2 806             16    .116   173.8   191.9    2.58    5.66
E1 770   2             20   10.15    2.27    38.1   26.22   3.189                 1.
E1 802   1              1    350.   3.500   .2097   .0209           1.0
E1   1 770             16
E1  94                 16    .644   173.8   191.9    2.58    5.66
E1  96                 16   .0388   173.8   191.9    2.58    5.66
E1  98 802   94   96   16
E1 772  98             16
E1 774 772             18           11.73                                       776.
E1 776 774             16
E1 150                 16    1.00   173.8   191.9    2.58    5.66
E1 155 365 150 220     16
E1 220                 16   .0171   1067.   2077.    19.4
E1 160 155 422         16
E1 860 160             16
E1  99 776             16
```

TABLE C 1.1 (Sheet 6 of 6)

```
E2
F1   1S.S.      MG/L     0
F1   2BOD5      MG/L     0
F1   3SOL.P     MG/L     0
F1   4TOTAL-P   MG/L     0
I1 24  160    6  772
J2330  340  350  363  365  410  422  510  522    2    1  772  776
R1     16.0      0.0
R1     16.0      0.0
R1     16.0      0.0
R1     16.0      0.0
R1     20.5      4.44
R1     20.5      3.79
R1     20.5      4.61
R1     20.5      3.62
R1     26.0      0.0
R1     26.0      0.0
R1     26.0      0.0
R1     26.0      0.0
ENDPROGRAM
```

# APPENDIX C2

## DERIVATION OF RUNOFF DATA

TABLE C 2.1

FLOW RATES AND SS LOAD RATES OF RUNOFF

| Date & Time | Flow Rate Q | SS Load Rate | Date & Time | Flow Rate Q | SS Load Rate |
|---|---|---|---|---|---|
| 830519 | | | 830529 | | |
| 14:15 | 1.616 | 343764 | 14:40 | 1.425 | 474089 |
| 14:45 | 3.913 | 1059980 | 15:10 | 4.382 | 1533500 |
| 15:15 | 3.993 | 1055670 | 15:40 | 3.729 | 796316 |
| 15:45 | 2.403 | 368720 | 16:10 | 0.966 | 94712 |
| 16:15 | 1.946 | 95519 | 16:40 | 1.680 | 107122 |
| 16:45 | 2.473 | 118410 | | | |
| 17:25 | 1.620 | 14843 | | | |
| 18:05 | 1.883 | 25966 | | | |
| 830606 | | | 830704 | | |
| 09:15 | 0.887 | 180046 | 20:00 | 1.019 | 103145 |
| 10:00 | 1.585 | 304757 | 20:15 | 1.556 | 158828 |
| 10:45 | 1.556 | 170036 | 20:30 | 1.712 | 196700 |
| 11:30 | 1.687 | 151346 | 20:45 | 1.325 | 137907 |
| 12:00 | 1.453 | 107020 | | | |
| 830808 | | | 830822 | | |
| 18:20 | 6.700 | 1823710 | 03:25 | 2.286 | 1131530 |
| 18:50 | 1.804 | 141408 | 04:25 | 2.560 | 1672510 |
| 19:20 | 1.543 | 320430 | 05:25 | 3.678 | 362360 |
| 19:50 | 0.912 | 107069 | 06:25 | 1.855 | 192168 |
| | | | 07:50 | 1.091 | 6517 |
| 830916 | | | 831013 | | |
| 08:10 | 1.362 | 415616 | 15:00 | 2.589 | 1446480 |
| 08:55 | 1.294 | 125816 | 16:00 | 2.194 | 186590 |
| 09:40 | 1.518 | 117440 | 17:00 | 2.589 | 131171 |
| 10:30 | 1.545 | 135158 | | | |
| 11:10 | 1.446 | 109808 | | | |
| 11:50 | 1.164 | 348548 | | | |

Loadrate = 77379 * Q^1.66
Where Loadrate is in mg/s; Q is in m^3/s
Coefficient of Correlation = 0.62
Notes: (1) Derived from station 1 and station 3 data.
(2) Runoff = Combined sewage less sanitary sewage.

TABLE C 2.2

FLOW RATES AND BOD5 LOAD RATES OF RUNOFF (1)

| Date & Time | Flow Rate Q | BOD5 Load Rate | Date & Time | Flow Rate Q | BOD5 Load Rate |
|---|---|---|---|---|---|
| 830519 | | | 830529 | | |
| 14:15 | 1.616 | 116076 | 15:10 | 4.38 | 320695 |
| 14:45 | 3.913 | 196794 | 15:40 | 3.72 | 190235 |
| 15:15 | 3.993 | 94129 | 16:40 | 1.68 | 8150 |
| 15:45 | 2.403 | 2176 | | | |
| 830606 | | | 830704 | | |
| 09:15 | 0.887 | 58341 | 20:00 | 1.01 | 2725 |
| 10:00 | 1.585 | 68203 | 20:15 | 1.55 | 43188 |
| 10:45 | 1.556 | 6949 | 20:30 | 1.71 | 46164 |
| | | | 20:45 | 1.32 | 29790 |
| 830808 | | | 830822 | | |
| 18:20 | 6.700 | 237763 | 03:25 | 2.28 | 319348 |
| 18:50 | 1.804 | 8481 | 04:25 | 2.56 | 431700 |
| 19:20 | 1.543 | 11827 | 05:25 | 3.67 | 37786 |
| 830916 | | | 831012 | | |
| 08:10 | 1.362 | 120658 | 05:15 | 1.44 | 184764 |
| 08:55 | 1.294 | 690 | 05:45 | 1.47 | 202594 |
| 09:40 | 1.518 | 8044 | 06:15 | 1.36 | 55372 |
| 10:30 | 1.545 | 5410 | | | |
| 11:50 | 1.164 | 111964 | | | |
| 831013 | | | | | |
| 15:00 | 2.589 | 210439 | | | |
| 16:00 | 2.194 | 70168 | | | |
| 17:00 | 2.589 | 1567 | | | |
| 18:00 | 1.135 | 4300 | | | |

********************************
* Loadrate = 13736 * Q^1.50 *
* Where Loadrate is in mg/s *
* Q is in m3/s *
* Coefficient of *
* Correlation = 0.40 *
* Notes:(1) Derived from st.1*
* and 3 data.(2) Runoff=Comb.*
* sewage less sanitary sewage*
********************************

TABLE C2.3

POLLUTANT RATIOS
TOTAL PHOSPHOROUS vs SUSPENDED SOLIDS IN RUNOFF (1)

| Date & Time | TP (ug/l) | SS (mg/l) | Date & Time | TP (ug/l) | SS (mg/l) |
|---|---|---|---|---|---|
| 830519 | | | 830529 | | |
| 14:15 | 3.1 | 212.7 | 14:40 | 5.0 | 332.7 |
| 14:45 | 2.1 | 270.9 | 15:10 | 2.5 | 349.9 |
| 15:15 | 1.3 | 264.4 | 15:40 | 1.4 | 213.5 |
| 15:45 | 0.5 | 153.4 | 16:10 | 0.2 | 98.0 |
| 16:15 | | | 16:40 | 0.6 | 63.8 |
| 16:45 | | | | | |
| 17:25 | | | | | |
| 18:05 | | | | | |
| 830606 | | | 830704 | | |
| 0915 | 3.2 | 203.0 | 20:00 | 0.6 | 101.2 |
| 1000 | 2.2 | 192.3 | 20:15 | 1.0 | 102.1 |
| 1045 | 0.9 | 109.3 | 20:30 | 1.2 | 114.9 |
| 1130 | 0.6 | 89.7 | 20:45 | 1.0 | 104.1 |
| 1200 | 0.5 | 73.6 | | | |
| 830808 | | | 830822 | | |
| 18:20 | 2.2 | 272.2 | 03:25 | 2.9 | 495.0 |
| 18:50 | 1.1 | 78.4 | 04:25 | 5.0 | 653.3 |
| 19:20 | 1.5 | 207.7 | | | |
| 19:50 | | | | | |
| 830916 | | | 831013 | | |
| 08:10 | 4.2 | 305.1 | 15:00 | 3.2 | 558.7 |
| 08:55 | 1.2 | 97.1 | 16:00 | 0.2 | 85.0 |
| 09:40 | 0.7 | 77.4 | 17:00 | 0.3 | 50.7 |
| 10:30 | 0.5 | 87.5 | 18:00 | | |
| 11:10 | 1.9 | 75.9 | | | |
| 11:50 | 4.6 | 299.4 | | | |

Mean Ratio = 9.172 ug/mg
Std. Dev. = 4.704 ug/mg

Notes: (1) Derived from Station 1 and Station 3 data.
Runoff Conc. = (comb. sewage load rate-san. sewage load rate)/
(comb. sewage flow rate-san. sewage flow rate)

TABLE C2.4

POLLUTANT RATIOS
LEAD vs SUSPENDED SOLIDS IN RUNOFF (1)

| Date & Time | Lead (ug/l) | SS (mg/l) | Date & Time | Lead (ug/l) | SS (mg/l) |
|---|---|---|---|---|---|
| 830519 | | | 830704 | | |
| 14:15 | 280.0 | 218.0 | 20:00 | 170.0 | 135.0 |
| 14:45 | 290.0 | 269.0 | 20:15 | 170.0 | 126.0 |
| 15:15 | 160.0 | 263.0 | 20:30 | 180.0 | 135.0 |
| 15:45 | 90.0 | 164.0 | 20:45 | 180.0 | 131.0 |
| 16:15 | 60.0 | 76.2 | | | |
| 16:45 | 60.0 | 70.0 | | | |
| 17:25 | 60.0 | 47.3 | | | |
| 18:05 | 60.0 | 46.7 | | | |
| 830808 | | | 830822 | | |
| 18:20 | 500.0 | 271.0 | 03:25 | 530.0 | 465.0 |
| 18:50 | 410.0 | 103.0 | 04:25 | 420.0 | 609.0 |
| 19:20 | 430.0 | 214.0 | 05:25 | 110.0 | 110.0 |
| 19:50 | 380.0 | 150.0 | 06:25 | 70.0 | 124.0 |
| | | | 07:25 | 50.0 | 44.1 |
| | | | 07:50 | 40.0 | 59.2 |
| 830916 | | | 831013 | | |
| 08:10 | 220.0 | 294.0 | 15:00 | 230.0 | 525.0 |
| 08:55 | 130.0 | 126.0 | 16:00 | 130.0 | 105.0 |
| 09:40 | 80.0 | 106.0 | 17:00 | 100.0 | 71.6 |
| 10:30 | 160.0 | 114.0 | 18:00 | 80.0 | 50.3 |
| 11:10 | 90.0 | 106.0 | | | |
| 11:50 | 220.0 | 288.0 | | | |

Mean Ratio = 1.125 ug/mg
Std Dev. = 0.676 ug/mg

Notes: (1) Derived from Station 1 and Station 3 data.
Runoff Conc. = (comb. sewage load rate-san. sewage load rate)/
(comb. sewage flow rate-san. sewage flow rate)

TABLE C 2.5

POLLUTANT RATIOS
ZINC vs SUSPENDED SOLIDS IN RUNOFF (1)

| Date & Time | Zinc (ug/l) | SS (mg/l) | Date & Time | Zinc (ug/l) | SS (mg/l) |
|---|---|---|---|---|---|
| 830519 | | | 830704 | | |
| 14:15 | 463 | 260.1 | 20:00 | 331 | 176.3 |
| 14:45 | 405 | 290.4 | 20:15 | 272 | 151.3 |
| 15:15 | 254 | 283.5 | 20:30 | 295 | 159.6 |
| 15:45 | 177 | 185.3 | 20:45 | 373 | 161.8 |
| 16:15 | 142 | 88.4 | | | |
| 16:45 | 121 | 78.8 | | | |
| 17:25 | 93 | 56.4 | | | |
| 18:05 | 107 | 54.4 | | | |
| 830808 | | | 830822 | | |
| 18:20 | 734 | 283.6 | 03:25 | 791 | 528.4 |
| 18:50 | 634 | 120.8 | 04:25 | 806 | 683.2 |
| 19:20 | 945 | 257.3 | 05:25 | 178 | 119.3 |
| 19:50 | 750 | 201.3 | 06:25 | 142 | 144.8 |
| | | | 07:25 | 151 | 54.4 |
| | | | 07:50 | 123 | 76.1 |
| 830916 | | | 831013 | | |
| 08:10 | 507 | 361.3 | 15:00 | 413 | 588.2 |
| 08:55 | 312 | 156.4 | 16:00 | 177 | 119.9 |
| 09:40 | 248 | 127.8 | 17:00 | 133 | 80.2 |
| 10:30 | 284 | 137.0 | 18:00 | 98 | 64.1 |
| 11:10 | 224 | 128.9 | | | |
| 11:50 | 542 | 365.2 | | | |

Mean Ratio = 1.886 ug/mg
Std. Dev. = 0.909 ug/mg

Notes:(1)Derived from station 1 and station 3 data.
(2)Runoff Conc.=(Comb.sewage load rate - San.sewage load rate.)/
(Comb.sewage flow rate - San.sewage flow rate.)

TABLE C 2.6

POLLUTANT RATIOS
COPPER vs SUSPENDED SOLIDS IN RUNOFF (1)

| Date & Time | Copper (ug/l) | SS (mg/l) | Date & Time | Copper (ug/l) | SS (mg/l) |
|---|---|---|---|---|---|
| 830519 | | | 830704 | | |
| 14:15 | 160 | 260.1 | 20:00 | 108 | 176.3 |
| 14:45 | 182 | 290.4 | 20:15 | 76 | 151.3 |
| 15:15 | 95 | 283.5 | 20:30 | 113 | 159.6 |
| 15:45 | 58 | 185.3 | 20:45 | 382 | 161.8 |
| 16:15 | 32 | 88.4 | | | |
| 16:45 | 36 | 78.8 | | | |
| 17:25 | 41 | 56.4 | | | |
| 18:05 | 43 | 54.4 | | | |
| 830808 | | | 830822 | | |
| 18:20 | 129 | 283.6 | 03:25 | 331 | 528.4 |
| 18:50 | 125 | 120.8 | 04:25 | 295 | 683.2 |
| 19:20 | 124 | 257.3 | 05:25 | 62 | 119.3 |
| 19:50 | 93 | 201.3 | 06:25 | 32 | 114.8 |
| | | | 07:25 | 12 | 54.4 |
| | | | 07:50 | 19 | 76.1 |
| 830916 | | | 831013 | | |
| 08:10 | 361 | 258.3 | 15:00 | 160 | 588.2 |
| 08:55 | 156 | 172.4 | 16:00 | 46 | 119.9 |
| 09:40 | 128 | 147.9 | 17:00 | 37 | 80.2 |
| 10:30 | 137 | 160.0 | 18:00 | 20 | 64.1 |
| 11:10 | 129 | 111.4 | | | |
| 11:50 | 365 | 400.9 | | | |

Mean Ratio = 0.645 ug/mg
Std. Dev. = 0.422 ug/mg

Notes :(1) Derived from station 1 and station 3 data.
(2) Runoff Conc.= (Comb.sewage load rate - San.sewage load rate)/
(Comb.sewage flow rate - San.sewage flow rate)

TABLE C 2.7

POLLUTANT RATIOS
CADMIUM vs SUSPENDED SOLIDS IN RUNOFF (1)

| Date & Time | Cadmium (ug/l) | SS (mg/l) | Date & Time | Cadmium (ug/l) | SS (mg/l) |
|---|---|---|---|---|---|
| 830519 | | | 830704 | | |
| 14:15 | 9 | 260.0 | 20:00 | 2 | 176.2 |
| 14:45 | 10 | 290.4 | 20:15 | 3 | 151.2 |
| 15:15 | 10 | 283.5 | 20:30 | 3 | 159.5 |
| 15:45 | 10 | 185.3 | 20:45 | 3 | 161.8 |
| 16:15 | 9 | 88.4 | | | |
| 16:45 | 10 | 78.8 | | | |
| 17:25 | 9 | 56.4 | | | |
| 18:05 | 9 | 54.4 | | | |
| 830808 | | | 830822 | | |
| 18:20 | 5 | 283.6 | 03:25 | 3 | 528.4 |
| 18:50 | 3 | 120.8 | 04:25 | 4 | 683.2 |
| 19:20 | 3 | 257.2 | 05:25 | 3 | 119.3 |
| 19:50 | 2 | 201.2 | 06:25 | 2 | 144.8 |
| | | | 07:25 | 2 | 54.4 |
| | | | 07:50 | 1 | 76.1 |
| 830916 | | | 831013 | | |
| 08:10 | 4 | 361.3 | 15:00 | 4 | 588.2 |
| 08:55 | 4 | 156.3 | 16:00 | 4 | 119.9 |
| 09:40 | 4 | 127.7 | 17:00 | 4 | 80.2 |
| 10:30 | 4 | 137.0 | 18:00 | 3 | 64.1 |
| 11:10 | 4 | 128.8 | | | |
| 11:50 | 4 | 365.1 | | | |

Mean Ratio = 0.038 ug/mg
Std. Dev. = 0.042 ug/mg

Notes :(1) Derived from station 1 and station 3 data.
(2) Runoff Conc.= (Comb. sewage load rate - San. sewage load rate)/ (Comb. sewage flow rate - San. sewage flow rate)

# APPENDIX C3

## CALIBRATION HYDROGRAPHS

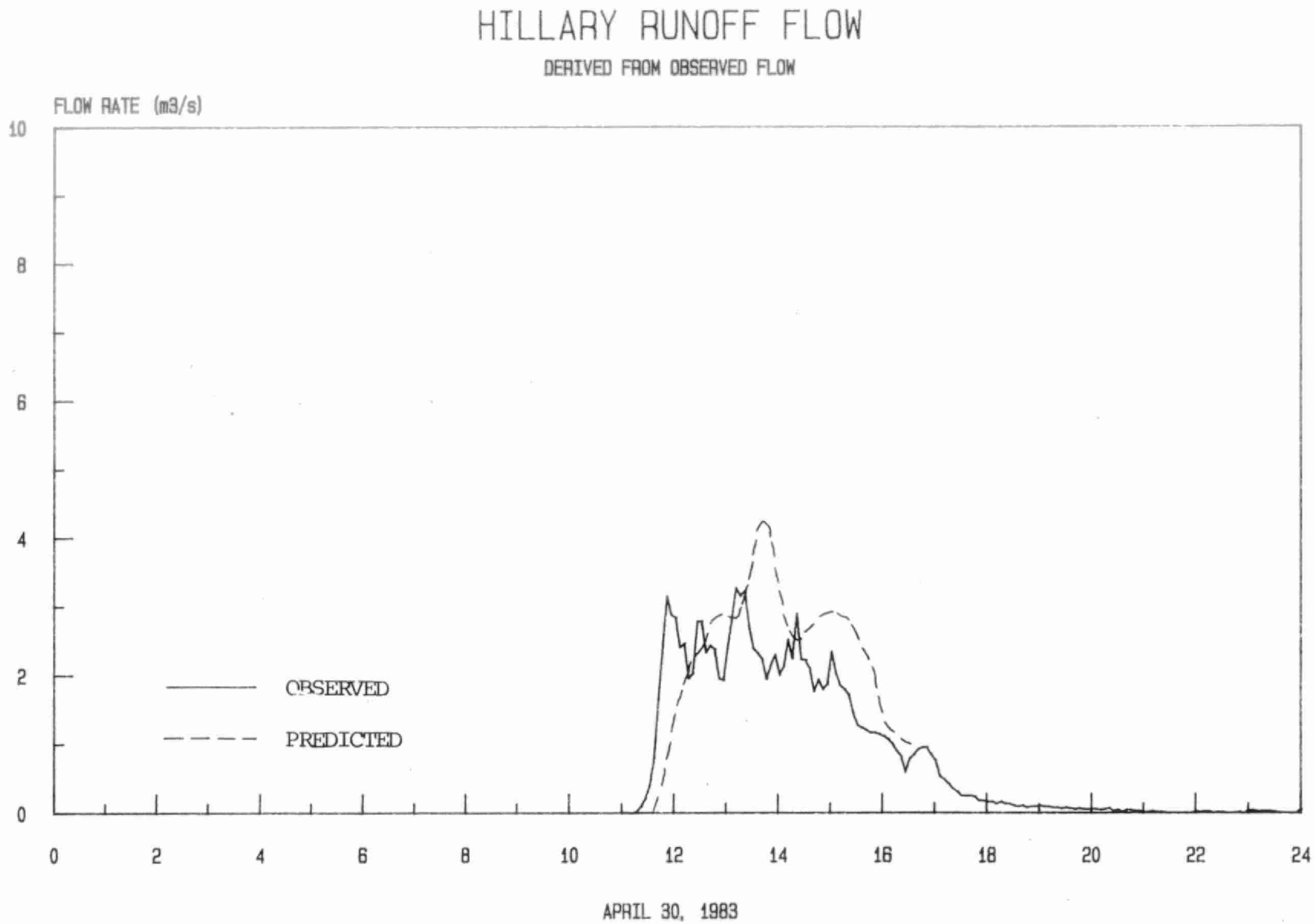

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
OBSERVED
PREDICTED
0
2
4
6
8
10
12
14
16
18
20
22
24
APRIL 30, 1983

FIGURE C3.1

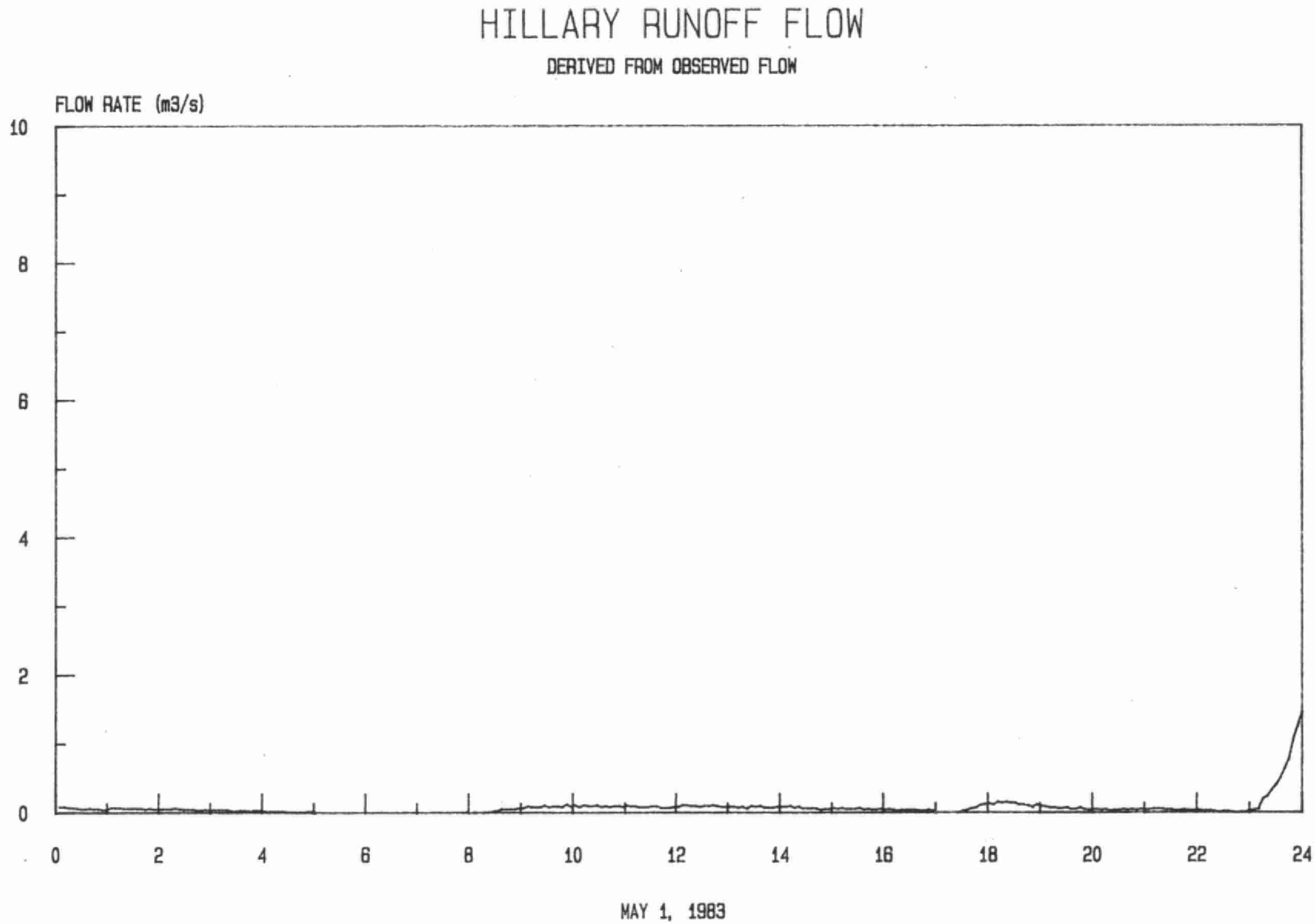

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
0
2
4
6
8
10
12
14
16
18
20
22
24
MAY 1, 1983

FIGURE C3.2

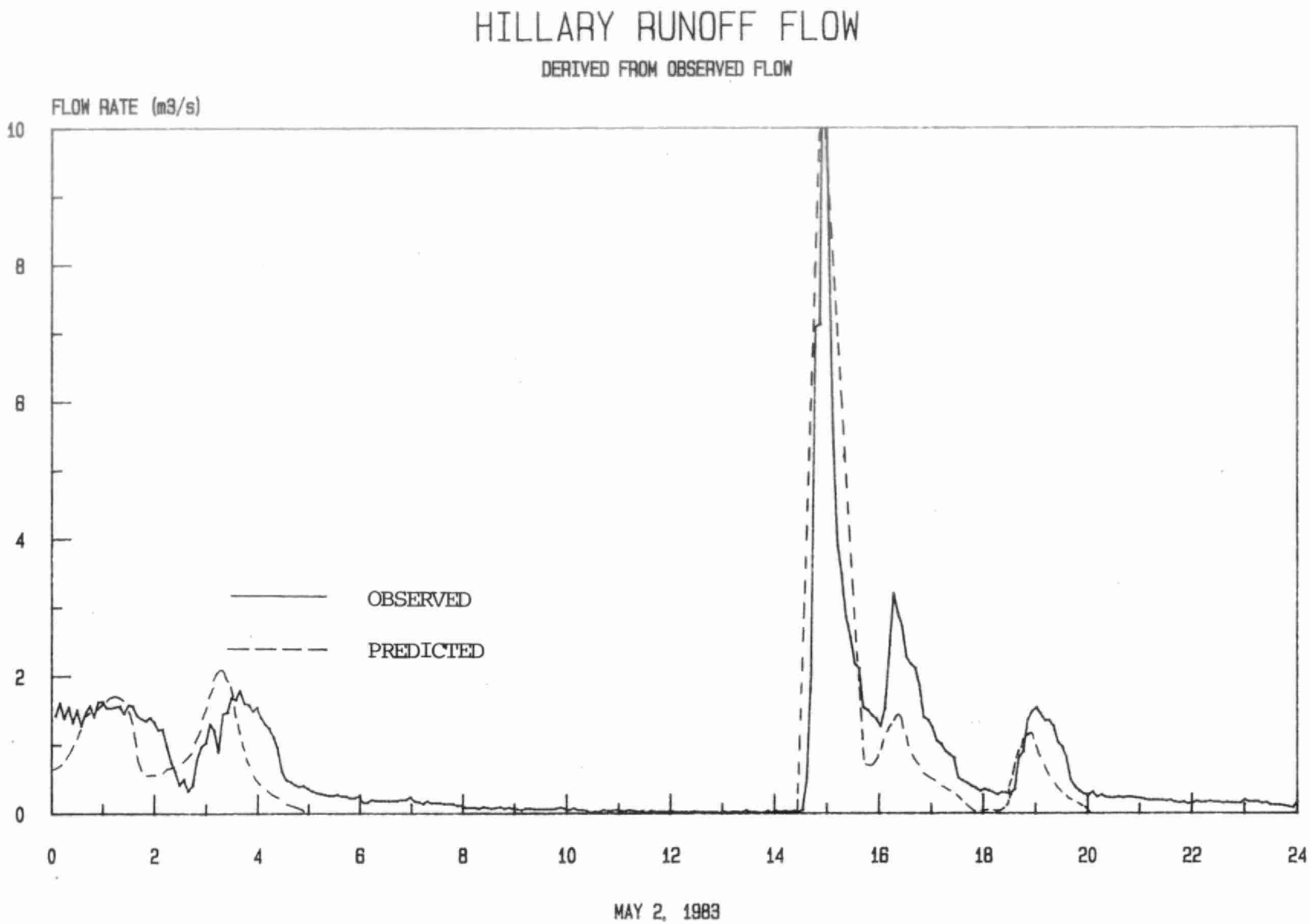

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
OBSERVED
PREDICTED
0
2
4
6
8
10
12
14
16
18
20
22
24
MAY 2, 1983

FIGURE C3.3

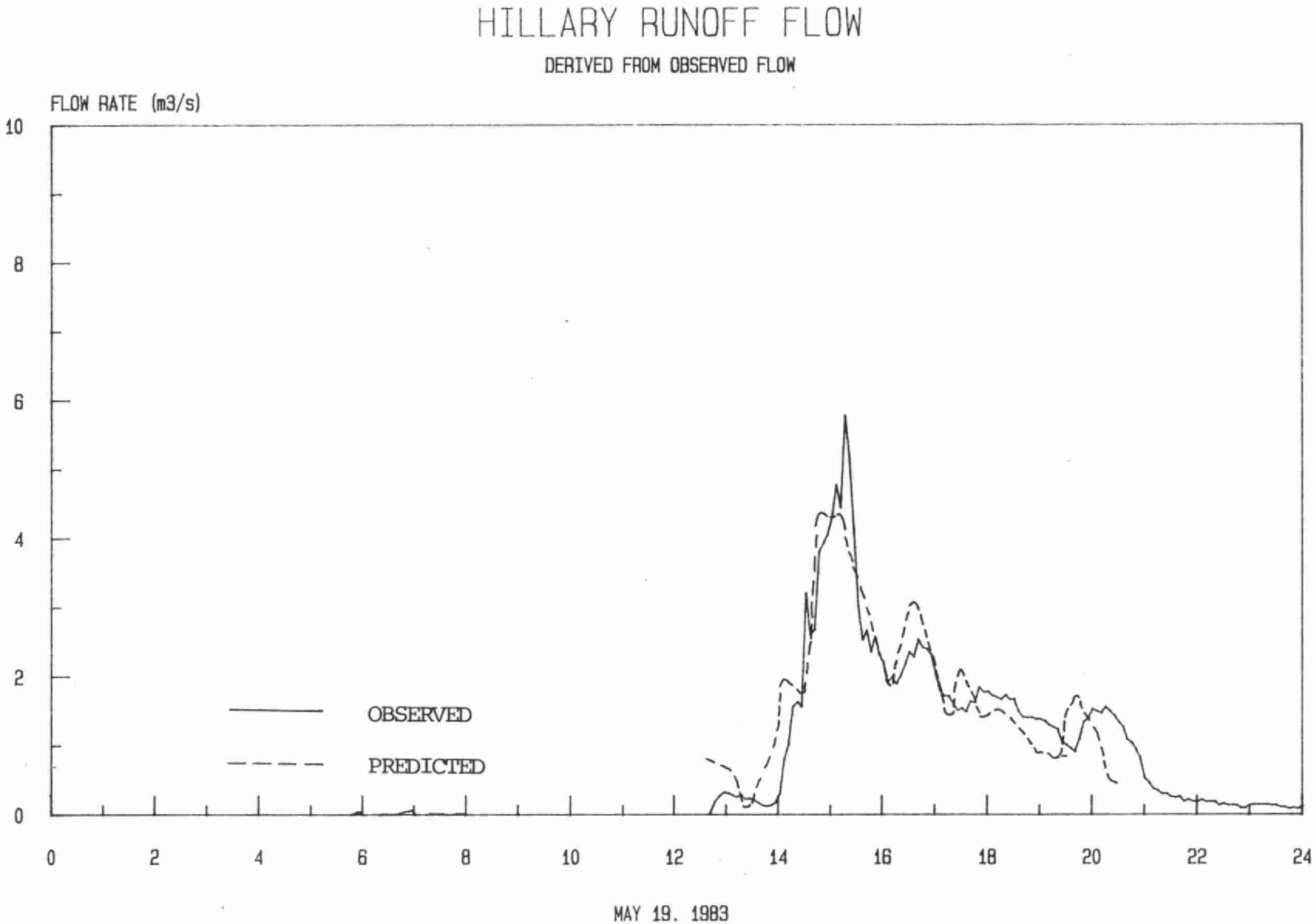

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
OBSERVED
PREDICTED
0
2
4
6
8
10
12
14
16
18
20
22
24
MAY 19. 1983

FIGURE C3.4

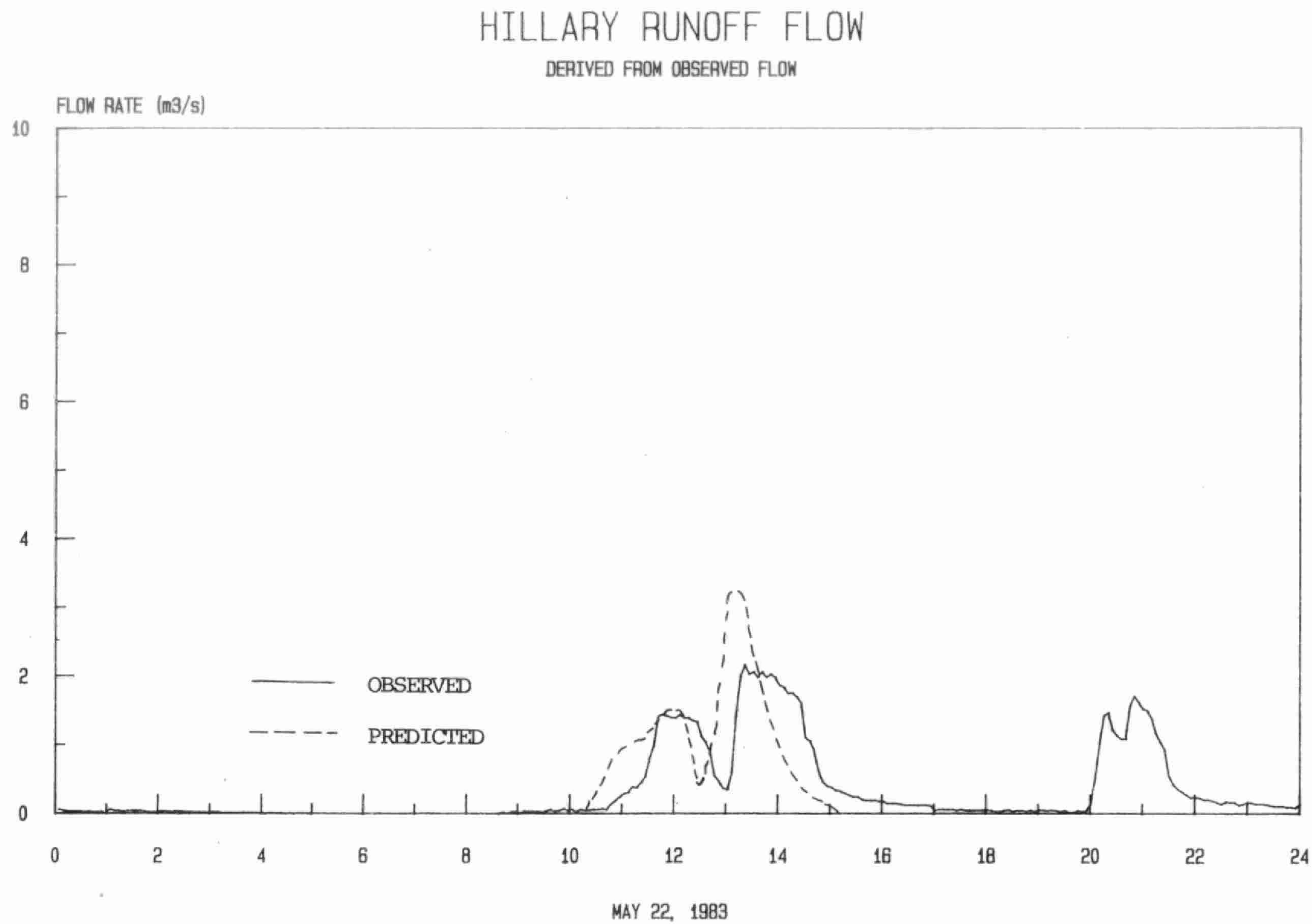

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
OBSERVED
PREDICTED
0
2
4
6
8
10
12
14
16
18
20
22
24
MAY 22, 1983

FIGURE C3.5

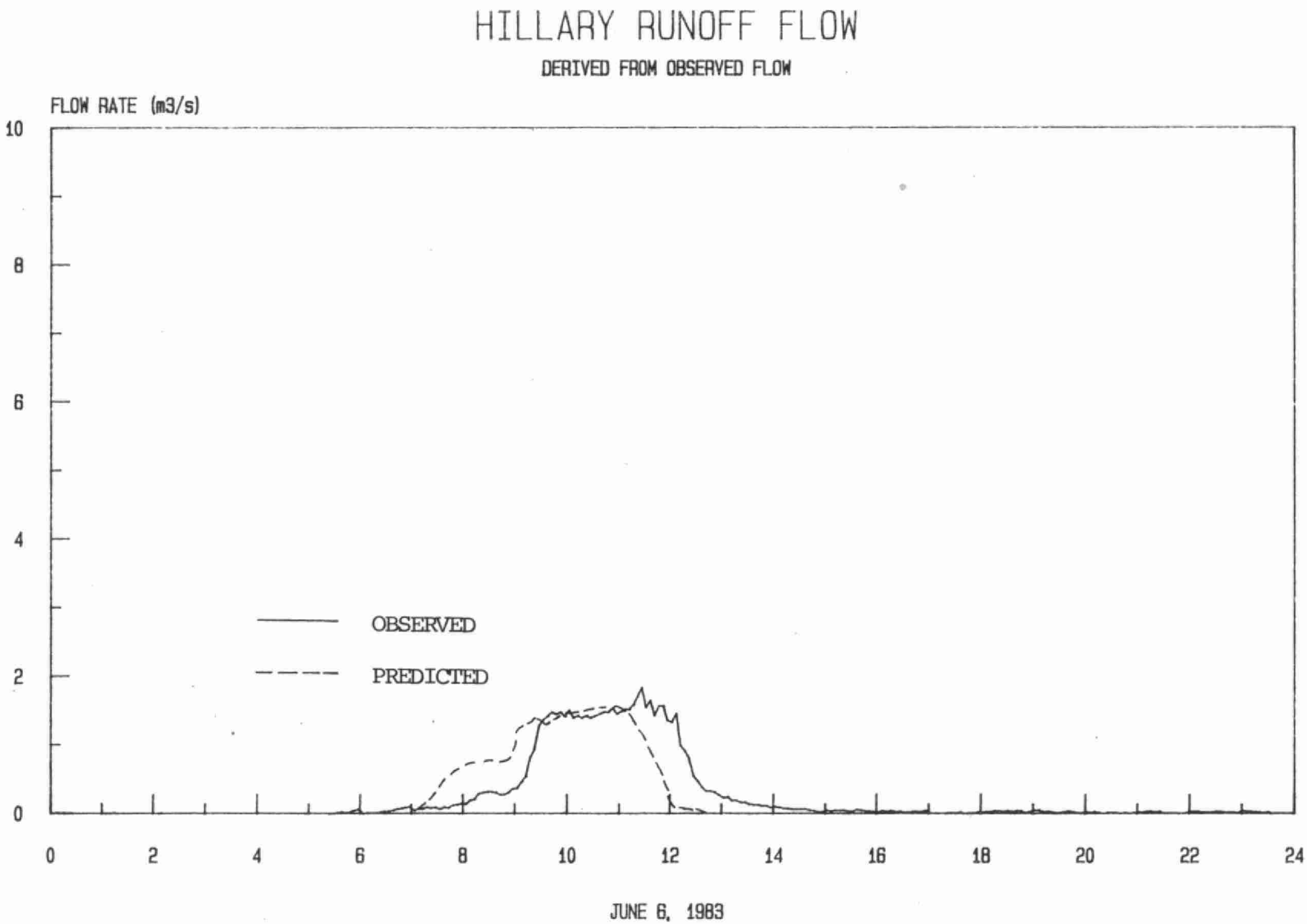

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
OBSERVED
PREDICTED
0
2
4
6
8
10
12
14
16
18
20
22
24
JUNE 6, 1983

FIGURE C3.6

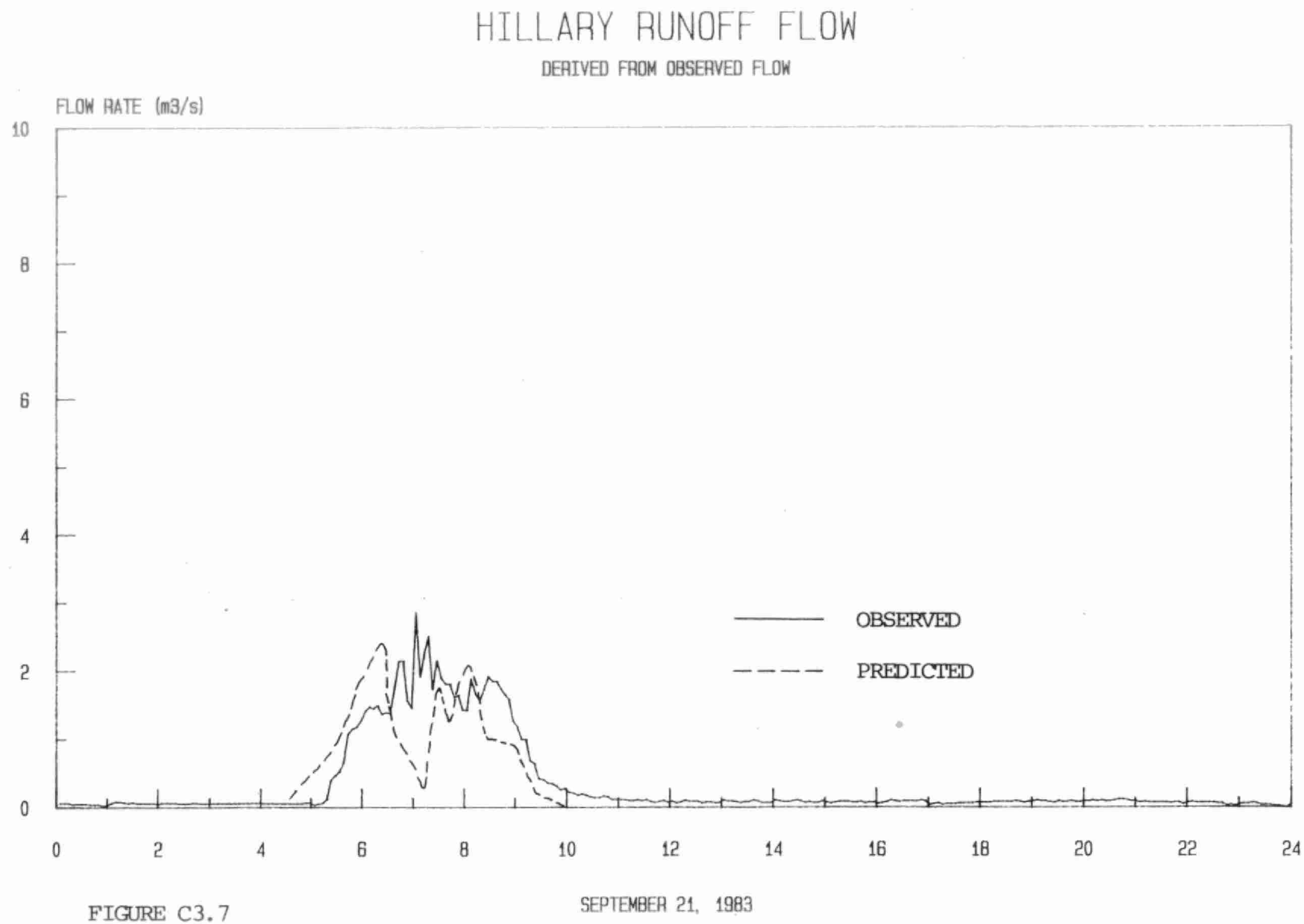

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
0
2
4
6
8
10
12
14
16
18
20
22
24
OBSERVED
PREDICTED
SEPTEMBER 21, 1983

FIGURE C3.7

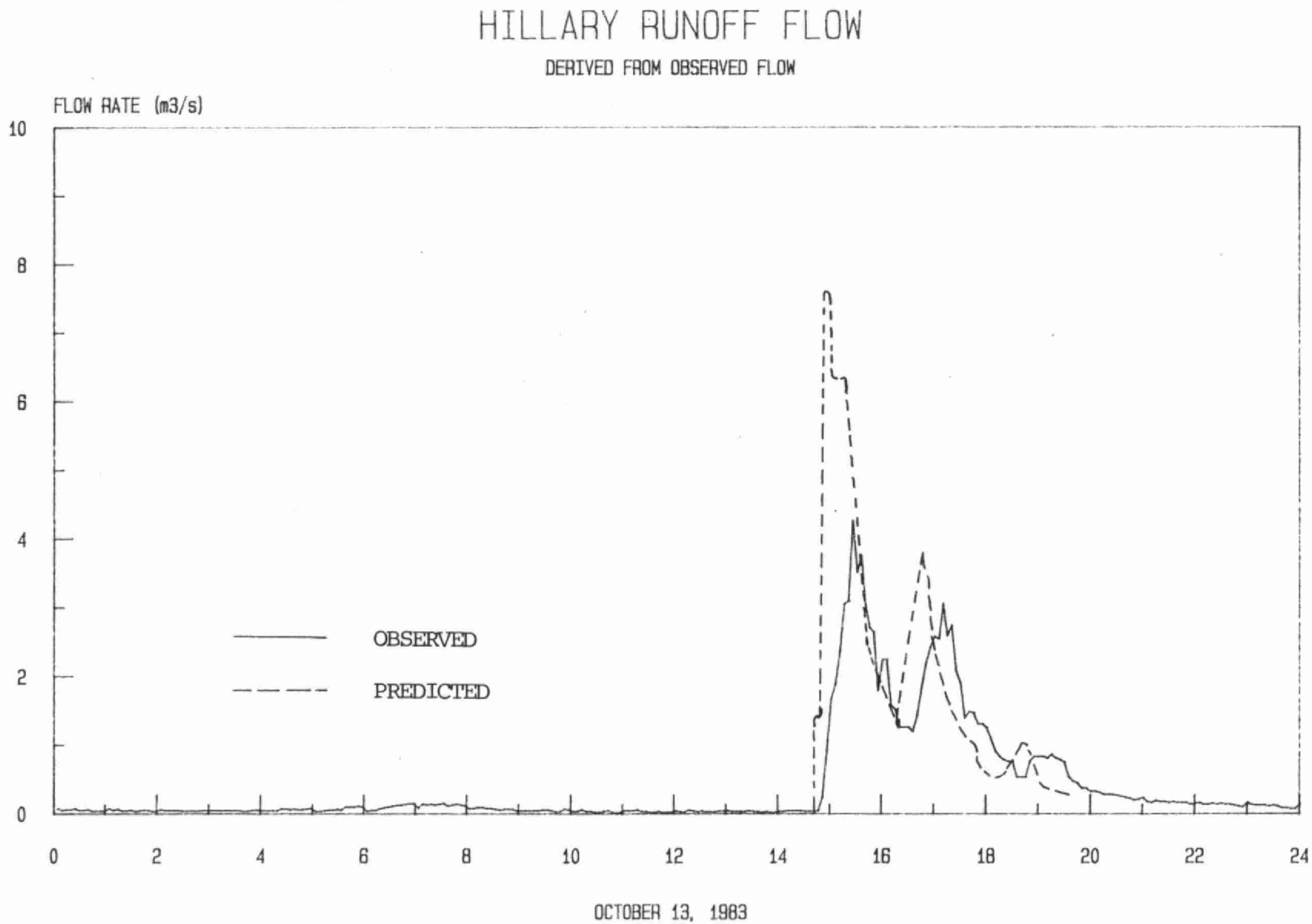

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
0
2
4
6
8
10
12
14
16
18
20
22
24
OCTOBER 13, 1983
OBSERVED
PREDICTED

FIGURE C3.8

# APPENDIX D1

## BASE CASE RESULTS

TABLE D1.1

SUMMARY OF RESULTS FOR APRIL - OCTOBER 1979
EXISTING SYSTEM

| | MHID | New+Old Tank Size | Volume m^3 | SS kg | BOD5 kg | SOL-P kg | Tp kg | Cd kg | Cu kg | Pb kg | Zn kg |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Overflow | BC Gp | | 493145 | 96554 | 23150 | 324 | 1015 | 3.975 | 61.088 | 101.070 | 175.584 |
| Stored | Hyde | 7823 | 110005 | 15943 | 4655 | 75 | 211 | .816 | 12.753 | 20.180 | 34.927 |
| Overflow | 660 | | 11974 | 523 | 556 | 8 | 15 | .039 | .960 | .140 | .510 |
| Overflow | 670 | | 92652 | 4214 | 4335 | 62 | 122 | .310 | 7.780 | 1.280 | 4.320 |
| Overflow | 6361 | | 384718 | 80343 | 18370 | 244 | 808 | 3.223 | 50.176 | 85.753 | 147.880 |
| Stored | | 8823 | 216421 | 32156 | 9667 | 155 | 386 | 1.472 | 22.800 | 35.389 | 61.687 |
| Overflow | 6362 | | 331356 | 71267 | 15782 | 207 | 704 | 2.822 | 43.798 | 81.411 | 130.293 |
| Stored | | 12823 | 544745 | 41232 | 12066 | 192 | 490 | 1.872 | 29.178 | 31.774 | 79.275 |
| Overflow | 6363 | | 291356 | 64111 | 13974 | 181 | 624 | 2.508 | 38.741 | 89.080 | 116.202 |
| Stored | | 16823 | 309784 | 48389 | 13873 | 218 | 570 | 2.186 | 34.236 | 24.510 | 93.365 |
| Overflow | 6364 | | 255168 | 57257 | 12322 | 157 | 550 | 2.211 | 33.922 | 96.208 | 102.702 |
| Stored | | 20823 | 345972 | 55242 | 15526 | 242 | 644 | 2.484 | 39.055 | 18.224 | 106.860 |
| Overflow | 6365 | | 221572 | 50649 | 10770 | 135 | 481 | 1.939 | 29.533 | 52.156 | 90.416 |
| Stored | | 24823 | 379567 | 61850 | 17078 | 264 | 713 | 2.755 | 43.444 | 68.988 | 119.151 |
| Overflow | 6366 | | 193903 | 45170 | 9490 | 117 | 430 | 1.742 | 26.425 | 47.185 | 81.856 |
| Stored | | 28823 | 407236 | 67330 | 18358 | 282 | 764 | 2.952 | 46.552 | 73.959 | 127.711 |
| Overflow | 6367 | | 175623 | 41487 | 8598 | 105 | 394 | 1.599 | 24.150 | 43.470 | 75.487 |
| Stored | | 32823 | 425516 | 71012 | 19206 | 294 | 801 | 3.096 | 48.827 | 77.673 | 134.081 |
| Overflow | 6368 | | 127623 | 31005 | 6295 | 75 | 290 | 1.185 | 17.549 | 32.397 | 56.550 |
| Stored | | 44823 | 473516 | 81494 | 21509 | 324 | 904 | 3.509 | 55.429 | 88.746 | 153.069 |
| Overflow | 6369 | | 116493 | 28379 | 5746 | 68 | 269 | 1.106 | 16.341 | 30.407 | 53.069 |
| Stored | | 48823 | 601139 | 84119 | 22058 | 331 | 924 | 3.588 | 56.636 | 90.737 | 156.497 |

Note: Event 790711 included.Tank size in m3.

TABLE -D1.2

EVENT 790711 RESULTS
EXISTING SYSTEM

| | MHID | Old+New Tank Size | Volume m^3 | SS kg | BOD5 kg | SOL-P kg | Tp kg | Cd kg | Cu kg | Pb kg | Zn kg |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Overflow | BC Gp | | 157493 | 33914 | 7151 | 95 | 325 | 1.331 | 19.857 | 36.380 | 63.143 |
| Stored | Hyde | 7823 | 7823 | 193 | 106 | 3 | 3 | .011 | .145 | .165 | .274 |
| Overflow | 660 | | 11974 | 523 | 556 | 8 | 15 | .039 | .960 | .140 | .510 |
| Overflow | 670 | | 92650 | 4214 | 4335 | 62 | 122 | .310 | 7.780 | 1.280 | 4.320 |
| Overflow | 6361 | | 156493 | 33820 | 7067 | 92 | 324 | 1.325 | 19.789 | 36.359 | 63.065 |
| Stored | | 8823 | 8823 | 287 | 189 | 6 | 5 | .016 | .212 | .186 | .353 |
| Overflow | 6362 | | 152493 | 33635 | 6989 | 89 | 321 | 1.317 | 19.647 | 36.172 | 62.767 |
| Stored | | 12823 | 12823 | 471 | 268 | 9 | 8 | .024 | .354 | .373 | .651 |
| Overflow | 6363 | | 148493 | 33373 | 6899 | 87 | 318 | 1.305 | 19.452 | 35.882 | 62.310 |
| Stored | | 16823 | 16823 | 734 | 357 | 11 | 11 | .036 | .550 | .663 | 1.107 |
| Overflow | 6364 | | 144493 | 33023 | 6797 | 84 | 315 | 1 .290 | 9.203 | 35.486 | 61.680 |
| Stored | | 20823 | 20823 | 1084 | 460 | 14 | 14 | .051 | .799 | 1.059 | 1.738 |
| Overflow | 6365 | | 140493 | 32584 | 6680 | 82 | 310 | 1.272 | 18.905 | 34.991 | 60.877 |
| Stored | | 24823 | 24823 | 1523 | 577 | 16 | 19 | .069 | 1.097 | 1.554 | 2.540 |
| Overflow | 6366 | | 136493 | 32060 | 6551 | 80 | 305 | 1.251 | 18.563 | 34.406 | 59.913 |
| Stored | | 28823 | 28823 | 2047 | 706 | 19 | 24 | .090 | 1.439 | 2.139 | 3.505 |
| Overflow | 6367 | | 132493 | 31458 | 6410 | 77 | 299 | 1.227 | 18.183 | 33.742 | 58.799 |
| Stored | | 32823 | 32823 | 2648 | 847 | 21 | 30 | .114 | 1.819 | 2.803 | 4.619 |
| Overflow | 6368 | | 120493 | 29238 | 5925 | 70 | 278 | 1.139 | 16.843 | 31.329 | 54.670 |
| Stored | | 44823 | 44823 | 4869 | 1332 | 28 | 51 | .202 | 3.159 | 5.216 | 8.748 |
| Overflow | 6369 | | 116493 | 28379 | 5746 | 68 | 269 | 1.106 | 16.341 | 30.407 | 53.069 |
| Stored | | 48823 | 48823 | 5727 | 1511 | 30 | 59 | .235 | 3.661 | 6.138 | 10.348 |

Note:(1) Tank size in m3.

TABLE D1.3

FECAL COLIFORM LOADS - EXISTING SYSTEM

| Event Date | Overflow F. Coliform Load ($10^{12}$ Organisms) |
|---|---|
| 790413 | 276 |
| 790426 | 385 |
| 790503 | 130 |
| 790525 | 212 |
| 790610 | 670 |
| 790629 | 123 |
| 790710 | 87 |
| 790716 | 341 |
| 790725 | 288 |
| 790731 | 642 |
| 790801 | 340 |
| 790914 | 639 |
| 791008 | 141 |
| 790715 (1) | 81 |
| 790807 (1) | 135 |
| 790826 (1) | 85 |

NOTE:

(1) Storm 790715 represents 5 storms of 4-6 mm precip.
Storm 790807 represents 3 storms of 6-8 mm precip.
Storm 790826 represents 5 storms of 8-10 mm precip.
F.C. Load is for single storm.

## APPENDIX D2

## BREAKDOWN OF ESTIMATES

ORDER OF COSTS

Unit Costs (Braganza, 1985)

| | | |
|---|---|---|
| 1. | Bulk excavation and backfill | $ 2.7/$m^3$ |
| 2. | Extra over item (1) for cartaway | $ 5.0/$m^3$ |
| 3. | Reinforced concrete including reinforcing steel and formwork | $ 200/$m^3$ |
| 4. | Road pavement, wearing course, basecourse and subbase | $ 50/$m^2$ |
| 5. | Excavate, supply and lay 1.2m die concrete sewer pipe, 5m depth | $ 450/m |
| 6. | Excavate, supply and lay 2.0m doa concrete sewer pipe, 5m depth | $ 780/m |
| 7. | Local detention tank: | |

| | | |
|---|---|---|
| 2.5m dia pipe | $ 1,148/m | |
| Road pavement 5.5m wide | 275/m | |
| | $ 1,423/m | |
| 60m long pipe at $1,423 | $87,380 | |
| Access manhole (say) | 4,000 | |
| | $91,380 | $91,380/No. |

(1) 16,000 m³ Tank at Hyde Ave

| Item | Quantity | Unit | Rate | Amount |
|---|---|---|---|---|
| Excavate, cartaway | 26,820 | $m^3$ | 5.00 | 134,000 |
| Concrete | 3,952 | $m^3$ | 200.00 | 790,000 |
| Total of (1) | | | | 924,000 |

(2) 41,000 m³ Tank at Black Creek

| | | | | |
|---|---|---|---|---|
| Excavate, replace | 95,000 | $m^3$ | 2.70 | 257,000 |
| Extra over for cartaway | 43,000 | $m^3$ | 5.00 | 215,000 |
| Concrete | 8,766 | $m^3$ | 200.00 | 1,753,000 |
| Total of (2) | | | | 2,225,000 |

(3) 35,000 m³ Tank at Black Creek

| | | | | |
|---|---|---|---|---|
| Excavate, replace | 80,820 | $m^3$ | 2.70 | 218,000 |
| Extra over for cartaway | 41,450 | $m^3$ | 5.00 | 207,000 |
| Concrete | 7,289 | $m^3$ | 200.00 | 1,458,000 |
| Total of (3) | | | | 1,883,000 |

(4) 15,000 $m^3$ Tank at Black Creek

| | | | | |
|---|---|---|---|---|
| Excavate, replace | 37,539 | $m^3$ | 2.70 | 101,000 |
| Extra over for cartaway | 20,529 | $m^3$ | 5.00 | 103,000 |
| Concrete | 3,305 | $m^3$ | 200.00 | 661,000 |
| Total of (4) | | | | 865,000 |

(5) 4,000 $m^3$ Tank at Berry Road

Extrapolate from Hyde Ave Tank

924,000 x 4,000/16,000 = 222,000

| | | |
|---|---|---|
| Add 10% for small job | 22,000 | 244,000 |
| Total of (5) | | 244,000 |

(6) 2m dia Pipe between New Hyde Ave Tank and New Black Creek Tank

| | | | | |
|---|---|---|---|---|
| Excavate, supply and lay pipe | 1,500 m | | 780.00 | 1,170,000 |
| Road reinstatement, 7m wide | 10,500 | $m^2$ | 50.00 | 525,000 |
| Total of (6) | | | | 1,695,000 |

(7) 1.2 dia Pipe for NEW
Black Creek Sewer

| | | | |
|---|---|---|---|
| Excavate, supply and lay pipe | 2,100 m | 450.00 | 945,000 |
| Total of (7) | | | 945,000 |

# APPENDIX D3

## CSO CONTROL SCHEMES RESULTS

TABLE D3.1

REQUIRED CAPACITIES OF CONNECTOR SEWERS

| Connector Sewer (1) | | | | Required Capacity ($m^3/s$) |
|---|---|---|---|---|
| | P21 | P31 | P41 | 7.5 |
| P11 | P22 | P32 | P42 | 1.4 |
| P12 | P23 | P33 | P45 | 3.3 |
| P13 | P24 | P34 | P43 | 2.3 |
| P14 | P25 | P35 | | 7.0 |
| | | | P44 | 3.7 |
| | P26 | | | 14.5 |

NOTE:

(1) See Figure 7.1 for naming of connector sewers.

TABLE D 3.2

SUMMARY OF RESULTS FOR APRIL - OCTOBER 1979
1983 CATCHMENTS AND REGULATORS RESET

| | MHID | Old+New Tank Size | Volume m^3 | SS kg | BOD5 kg | SOL-P kg | TP kg | Cd kg | Cu kg | Pb kg | Zn kg |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Overflow | BC Gp | | 221154 | 44413 | 9266 | 134 | 444 | 1.778 | 29.006 | 47.692 | 83.970 |
| Stored | Hyde | 7823 | 110005 | 15874 | 4635 | 75 | 210 | 0.811 | 12.711 | 20.096 | 34.791 |
| Overflow | 660 | | 17035 | 790 | 788 | 12 | 22 | 0.057 | 1.382 | 0.255 | 0.824 |
| Overflow | 670 | | 114264 | 5746 | 5474 | 80 | 156 | 0.406 | 9.850 | 2.039 | 6.301 |
| Overflow | 6361 | | 207766 | 41489 | 8594 | 123 | 409 | 1.641 | 26.904 | 44.518 | 78.394 |
| Stored | | 8823 | 123395 | 18802 | 5363 | 86 | 232 | 0.897 | 14.228 | 22.718 | 39.165 |
| Overflow | 6362 | | 179869 | 36099 | 7379 | 106 | 350 | 1.405 | 23.039 | 38.105 | 67.496 |
| Stored | | 12823 | 151292 | 24193 | 6590 | 104 | 291 | 1.131 | 18.093 | 29.132 | 50.063 |
| Overflow | 6363 | | 157137 | 31310 | 6373 | 91 | 301 | 1.208 | 19.753 | 32.613 | 58.183 |
| Stored | | 16823 | 174024 | 28981 | 7610 | 118 | 339 | 1.329 | 21.379 | 34.624 | 59.377 |
| Overflow | 6364 | | 143546 | 28409 | 5788 | 84 | 271 | 1.090 | 17.789 | 29.332 | 52.611 |
| Stored | | 20823 | 187615 | 31882 | 8208 | 126 | 368 | 1.447 | 23.343 | 37.904 | 64.948 |
| Overflow | 6365 | | 131546 | 25809 | 5269 | 77 | 246 | 0.991 | 16.137 | 26.590 | 47.921 |
| Stored | | 24823 | 199615 | 34483 | 8632 | 133 | 393 | 1.545 | 24.995 | 40.646 | 69.638 |
| Overflow | 6366 | | 119546 | 23137 | 4736 | 69 | 221 | 0.886 | 14.403 | 23.688 | 42.946 |
| Stored | | 28823 | 211615 | 37154 | 9167 | 141 | 419 | 1.650 | 26.729 | 43.548 | 74.613 |
| Overflow | 6367 | | 109813 | 20917 | 4301 | 63 | 198 | 0.798 | 12.932 | 21.221 | 38.704 |
| Stored | | 32823 | 221347 | 39375 | 9601 | 146 | 442 | 1.739 | 28.201 | 46.016 | 78.855 |
| Overflow | 6368 | | 92180 | 17108 | 3554 | 53 | 163 | 0.654 | 10.578 | 17.305 | 31.781 |
| Stored | | 44823 | 238981 | 43183 | 10348 | 157 | 477 | 1.882 | 30.554 | 49.932 | 85.778 |
| Overflow | 6369 | | 88180 | 16334 | 3395 | 51 | 156 | 0.624 | 10.111 | 16.530 | 30.346 |
| Stored | | 48823 | 242981 | 43957 | 10507 | 159 | 484 | 1.912 | 31.021 | 50.707 | 87.214 |

Notes: (1) Value of Event 790711 included.
(2) Tank size in m3.

TABLE D 3.3

EVENT 790711 RESULTS
1983 CATCHMENTS & RESET REGULATORS

| | MHID | Old+New Tank Size | Volume m^3 | SS kg | BOD5 kg | SOL-P kg | TP kg | Cd kg | Cu kg | Pb kg | Zn kg |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Overflow | BC Gp | | 129180 | 21350 | 4548 | 76 | 205 | 0.819 | 13.350 | 21.570 | 39.510 |
| Stored | Hyde | 7823 | 7823 | 124 | 86 | 3 | 2 | 0.006 | 0.103 | 0.081 | 0.138 |
| Overflow | 660 | | 16838 | 771 | 772 | 12 | 22 | 0.056 | 1.355 | 0.246 | 0.799 |
| Overflow | 670 | | 109524 | 5293 | 5103 | 75 | 146 | 0.378 | 9.191 | 1.836 | 5.741 |
| Overflow | 6361 | | 128180 | 21261 | 4499 | 74 | 204 | 0.815 | 13.274 | 21.507 | 39.381 |
| Stored | | 8823 | 8823 | 213 | 136 | 6 | 4 | 0.011 | 0.177 | 0.153 | 0.257 |
| Overflow | 6362 | | 124180 | 21108 | 4444 | 72 | 202 | 0.808 | 13.150 | 21.353 | 39.127 |
| Stored | | 12823 | 12823 | 366 | 191 | 8 | 6 | 0.017 | 0.301 | 0.307 | 0.511 |
| Overflow | 6363 | | 120180 | 20878 | 4378 | 69 | 200 | 0.799 | 12.977 | 21.116 | 38.724 |
| Stored | | 16823 | 16823 | 596 | 257 | 11 | 8 | 0.026 | 0.474 | 0.544 | 0.915 |
| Overflow | 6364 | | 116180 | 20563 | 4299 | 67 | 196 | 0.786 | 12.754 | 20.792 | 38.154 |
| Stored | | 20823 | 20823 | 911 | 337 | 13 | 11 | 0.039 | 0.697 | 0.868 | 1.484 |
| Overflow | 6365 | | 112180 | 20160 | 4205 | 65 | 192 | 0.771 | 12.485 | 20.381 | 37.418 |
| Stored | | 24823 | 24823 | 1315 | 430 | 15 | 15 | 0.054 | 0.966 | 1.279 | 2.220 |
| Overflow | 6366 | | 108180 | 19672 | 4097 | 62 | 188 | 0.752 | 12.173 | 19.889 | 36.524 |
| Stored | | 28823 | 28823 | 1802 | 538 | 18 | 20 | 0.073 | 1.278 | 1.771 | 3.114 |
| Overflow | 6367 | | 104180 | 19123 | 3977 | 60 | 182 | 0.731 | 11.822 | 19.332 | 35.510 |
| Stored | | 32823 | 32823 | 2352 | 658 | 20 | 26 | 0.094 | 1.629 | 2.328 | 4.128 |
| Overflow | 6368 | | 92180 | 17108 | 3554 | 53 | 163 | 0.654 | 10.578 | 17.305 | 31.781 |
| Stored | | 44823 | 44823 | 4366 | 1081 | 27 | 45 | 0.171 | 2.873 | 4.355 | 7.857 |
| Overflow | 6369 | | 88180 | 16334 | 3395 | 51 | 156 | 0.624 | 10.111 | 16.530 | 30.346 |
| Stored | | 48823 | 48823 | 5140 | 1240 | 29 | 52 | 0.201 | 3.340 | 5.130 | 9.293 |

Note: Tank size in m3.

TABLE D 3.4

SUMMARY OF RESULTS FOR APRIL - OCTOBER 1979
RUNOFF CONTROL SCHEME A

| | MHID | Old+New Tank Size | Volume m^3 | SS kg | BOD5 kg | SOL-P kg | TP kg | Cd kg | Cu kg | Pb kg | Zn kg |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Overflow | BC Gp | | 280730 | 44509 | 11696 | 184.6 | 469.9 | 1.806 | 28.270 | 43.810 | 76.859 |
| Stored | Hyde | 7823 | 109079 | 13515 | 4355 | 77.2 | 194.1 | 0.740 | 11.452 | 17.655 | 30.751 |
| Overflow | 660 | | 11269 | 493 | 531 | 7.7 | 14.7 | 0.036 | 0.918 | 0.125 | 0.476 |
| Overflow | 670 | | 90257 | 4037 | 4260 | 60.7 | 119.7 | 0.302 | 7.593 | 1.127 | 4.011 |
| Overflow | 6361 | | 223720 | 36503 | 8979 | 248.6 | 359.4 | 1.416 | 22.636 | 36.674 | 63.311 |
| Stored | | 8823 | 166082 | 21523 | 7072 | 282.2 | 280.6 | 1.056 | 16.088 | 24.360 | 42.794 |
| Overflow | 6362 | | 192372 | 31301 | 7523 | 187.0 | 298.2 | 1.179 | 19.019 | 30.721 | 53.058 |
| Stored | | 12823 | 201430 | 26725 | 8528 | 343.7 | 341.8 | 1.293 | 19.850 | 30.314 | 53.046 |
| Overflow | 6363 | | 156372 | 26292 | 6188 | 131.6 | 242.3 | 0.963 | 15.562 | 25.206 | 43.594 |
| Stored | | 16823 | 233430 | 31733 | 9863 | 399.2 | 397.6 | 1.508 | 23.307 | 35.829 | 62.510 |
| Overflow | 6364 | | 134627 | 22863 | 17299 | 99.1 | 208.8 | 0.834 | 13.546 | 22.103 | 38.221 |
| Stored | | 20823 | 255175 | 35162 | 10751 | 431.6 | 431.2 | 1.636 | 25.322 | 38.933 | 67.884 |
| Overflow | 6365 | | 118627 | 20441 | 4661 | 79.7 | 187.5 | 0.752 | 12.261 | 20.145 | 34.818 |
| Stored | | 24823 | 271175 | 37585 | 11390 | 451.0 | 452.4 | 1.718 | 26.608 | 40.889 | 71.287 |
| Overflow | 6366 | | 105650 | 18403 | 4161 | 66.3 | 171.6 | 0.693 | 11.302 | 18.646 | 32.227 |
| Stored | | 28823 | 284152 | 39622 | 11890 | 464.4 | 468.4 | 1.778 | 27.567 | 42.389 | 73.877 |
| Overflow | 6367 | | 93894 | 16472 | 3705 | 55.9 | 158.0 | 0.636 | 10.399 | 17.208 | 29.745 |
| Stored | | 32823 | 295908 | 41554 | 12346 | 475.8 | 483.0 | 1.834 | 28.469 | 43.827 | 76.359 |
| Overflow | 6368 | | 79520 | 14078 | 3159 | 46.8 | 136.3 | 0.522 | 9.060 | 15.021 | 25.912 |
| Stored | | 44823 | 310283 | 43948 | 12892 | 483.9 | 503.7 | 1.919 | 29.809 | 46.013 | 80.193 |
| Overflow | 6369 | | 75520 | 13356 | 2997 | 44.4 | 129.3 | 0.523 | 8.602 | 14.257 | 24.584 |
| Stored | | 48823 | 314283 | 44669 | 13053 | 486.3 | 510.7 | 1.947 | 30.266 | 46.777 | 81.521 |

Note: (1) 1983 sewer system. Event 790711 included.
(2) Tank size in m3.

TABLE D 3.5

EVENT 790711 RESULTS
RUNOFF CONTROL SCHEME A (1)

| | MHID | Old+New Tank Size | Volume m^3 | SS kg | BOD5 kg | SOL-P kg | TP kg | Cd kg | Cu kg | Pb kg | Zn Kg |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Overflow | BC Gp | | 116520 | 18301 | 4248 | 69.6 | 179.9 | 0.721 | 11.888 | 19.439 | 33.483 |
| Stored | Hyde | 7823 | 7823 | 273 | 139 | 5.2 | 4.1 | 0.015 | 0.242 | 0.255 | 0.393 |
| Overflow | 660 | | 11269 | 493 | 531 | 7.7 | 14.7 | 0.036 | 0.918 | 0.125 | 0.476 |
| Overflow | 670 | | 90257 | 4037 | 4260 | 60.7 | 119.7 | 0.302 | 7.593 | 1.127 | 4.011 |
| Overflow | 6361 | | 115519 | 18330 | 4218 | 68.6 | 179.4 | 0.723 | 11.929 | 19.539 | 33.593 |
| Stored | | 8823 | 8823 | 245 | 169 | 6.2 | 4.6 | 0.015 | 0.199 | 0.155 | 0.283 |
| Overflow | 6362 | | 115519 | 18160 | 4149 | 66.0 | 177.2 | 0.715 | 11.785 | 19.363 | 33.321 |
| Stored | | 12823 | 12823 | 415 | 238 | 8.7 | 6.8 | 0.023 | 0.344 | 0.331 | 0.555 |
| Overflow | 6363 | | 107519 | 17911 | 4069 | 63.6 | 174.3 | 0.704 | 11.586 | 19.092 | 32.893 |
| Stored | | 16823 | 16823 | 664 | 318 | 11.2 | 9.6 | 0.033 | 0.542 | 0.602 | 0.983 |
| Overflow | 6364 | | 103519 | 17576 | 3975 | 61.1 | 170.8 | 0.690 | 11.337 | 18.728 | 32.300 |
| Stored | | 20823 | 20823 | 998 | 411 | 13.6 | 13.2 | 0.047 | 0.791 | 0.967 | 1.576 |
| Overflow | 6365 | | 99519 | 17159 | 3868 | 58.7 | 166.5 | 0.673 | 11.044 | 18.278 | 31.547 |
| Stored | | 24823 | 24823 | 1416 | 519 | 16.0 | 17.4 | 0.064 | 1.085 | 1.416 | 2.329 |
| Overflow | 6366 | | 95519 | 16664 | 3748 | 56.3 | 161.6 | 0.654 | 10.710 | 17.750 | 30.650 |
| Stored | | 28823 | 28823 | 1910 | 639 | 18.4 | 22.4 | 0.084 | 1.419 | 1.944 | 3.226 |
| Overflow | 6367 | | 91519 | 16103 | 3616 | 53.9 | 156.0 | 0.631 | 10.339 | 17.152 | 29.625 |
| Stored | | 32823 | 32823 | 2472 | 771 | 20.8 | 28.0 | 0.106 | 1.789 | 2.543 | 4.251 |
| Overflow | 6368 | | 79520 | 14078 | 3159 | 46.8 | 136.3 | 0.552 | 9.060 | 15.021 | 25.912 |
| Stored | | 44823 | 44823 | 4497 | 1228 | 27.9 | 47.7 | 0.186 | 3.069 | 4.673 | 7.964 |
| Overflow | 6369 | | 75520 | 13356 | 2997 | 44.4 | 129.3 | 0.523 | 8.602 | 14.257 | 24.584 |
| Stored | | 48823 | 48823 | 5218 | 1389 | 30.3 | 54.7 | 0.214 | 3.526 | 5.437 | 9.292 |

Notes: (1) 1983 sewer system.
(2) Tank size in m3.

TABLE D 3.6

SUMMARY OF RESULTS FOR APRIL - OCTOBER 1979
RUNOFF CONTROL SCHEME B (1)

| | MHID | Old+New Tank Size | Volume m^3 | SS kg | BOD5 kg | SOL-P kg | TP kg | Cd kg | Cu kg | Pb kg | Zn Kg |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Overflow | BC Gp | | 230800 | 35698 | 9581 | 153 | 377 | 1.458 | 22.538 | 36.512 | 61.995 |
| Stored | Hyde | 7823 | 88367 | 10362 | 3468 | 63 | 153 | 0.576 | 8.858 | 13.496 | 23.510 |
| Overflow | 660 | | 11101 | 490 | 526 | 59 | 15 | 0.036 | .909 | 0.127 | 0.471 |
| Overflow | 670 | | 89518 | 4028 | 4245 | 158 | 119 | 0.303 | 7.531 | 1.155 | 4.041 |
| Overflow | 6361 | | 176357 | 28298 | 7032 | 92 | 279 | 1.105 | 17.559 | 29.799 | 49.597 |
| Stored | | 8823 | 142804 | 17762 | 6016 | 123 | 236 | 0.880 | 13.347 | 20.012 | 35.225 |
| Overflow | 6362 | | 144743 | 23686 | 5741 | 89 | 224 | 0.891 | 14.139 | 24.345 | 40.261 |
| Stored | | 12823 | 174417 | 22373 | 7307 | 126 | 291 | 1.094 | 16.737 | 25.468 | 44.561 |
| Overflow | 6363 | | 119885 | 19895 | 4731 | 73 | 183 | 0.738 | 11.730 | 20.552 | 33.770 |
| Stored | | 16823 | 199275 | 26164 | 8317 | 142 | 331 | 1.246 | 19.177 | 29.260 | 51.051 |
| Overflow | 6364 | | 105949 | 17778 | 4187 | 63 | 163 | 0.660 | 10.476 | 18.552 | 30.391 |
| Stored | | 20823 | 213211 | 28282 | 8861 | 152 | 351 | 1.325 | 20.430 | 31.260 | 54.430 |
| Overflow | 6365 | | 93949 | 15923 | 3735 | 56 | 149 | 0.604 | 9.594 | 17.118 | 27.983 |
| Stored | | 24823 | 225211 | 30136 | 9314 | 159 | 366 | 1.380 | 21.313 | 32.695 | 56.829 |
| Overflow | 6366 | | 82685 | 14140 | 3304 | 49 | 136 | 0.551 | 8.752 | 15.730 | 25.654 |
| Stored | | 28823 | 236475 | 31921 | 9745 | 166 | 379 | 1.433 | 22.155 | 34.081 | 59.167 |
| Overflow | 6367 | | 76367 | 13159 | 3072 | 45 | 128 | 0.521 | 8.296 | 14.966 | 24.377 |
| Stored | | 32823 | 242793 | 32901 | 9976 | 170 | 387 | 1.462 | 22.610 | 34.847 | 60.444 |
| Overflow | 6368 | | 64368 | 11069 | 2587 | 38 | 108 | 0.438 | 7.004 | 12.630 | 20.514 |
| Stored | | 44823 | 254793 | 34991 | 10461 | 177 | 408 | 1.545 | 23.902 | 37.183 | 64.308 |
| Overflow | 6369 | | 60368 | 10340 | 2419 | 36 | 100 | 0.410 | 6.551 | 11.807 | 19.163 |
| Stored | | 48823 | 258793 | 35720 | 10629 | 179 | 415 | 1.574 | 24.356 | 38.006 | 65.658 |

Notes: (1) 1983 sewer system. Event 790711 included.
(2) Tank size in m3.

TABLE D 3.7

EVENT 790711 RESULTS
RUNOFF CONTROL SCHEME B (1)

| | MHID | Old+New Tank Size | Volume m^3 | SS kg | BOD5 kg | SOL-P kg | TP kg | Cd kg | Cu kg | Pb kg | Zn Kg |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Overflow | BC Gp | | 101368 | 15640 | 3778 | 62 | 154 | 0.623 | 9.972 | 17.748 | 28.804 |
| Stored | Hyde | 7823 | 7823 | 354 | 161 | 5 | 5 | 0.017 | 0.304 | 0.405 | 0.556 |
| Overflow | 660 | | 11101 | 490 | 526 | 8 | 15 | 0.036 | 0.909 | 0.127 | 0.471 |
| Overflow | 670 | | 89518 | 4028 | 4245 | 60 | 119 | 0.303 | 7.531 | 1.155 | 4.014 |
| Overflow | 6361 | | 100367 | 15714 | 3762 | 60 | 154 | 0.625 | 10.040 | 17.927 | 29.006 |
| Stored | | 8823 | 8823 | 280 | 177 | 6 | 5 | 0.015 | 0.234 | 0.226 | 0.356 |
| Overflow | 6362 | | 96367 | 15501 | 3682 | 57 | 152 | 0.616 | 9.867 | 17.673 | 28.646 |
| Stored | | 12823 | 12823 | 493 | 257 | 9 | 7 | 0.024 | 0.407 | 0.481 | 0.716 |
| Overflow | 6363 | | 92367 | 15196 | 3587 | 55 | 148 | 0.603 | 9.639 | 17.306 | 28.107 |
| Stored | | 16823 | 16823 | 798 | 352 | 11 | 11 | 0.037 | 0.636 | 0.848 | 1.255 |
| Overflow | 6364 | | 88367 | 14802 | 3478 | 52 | 144 | 0.587 | 9.361 | 16.840 | 27.393 |
| Stored | | 20823 | 20823 | 1192 | 461 | 14 | 15 | 0.053 | 0.913 | 1.314 | 1.969 |
| Overflow | 6365 | | 84367 | 14324 | 3355 | 50 | 139 | 0.568 | 9.043 | 16.289 | 26.521 |
| Stored | | 24823 | 24823 | 1670 | 585 | 16 | 20 | 0.072 | 1.232 | 1.865 | 2.841 |
| Overflow | 6366 | | 80367 | 13774 | 3219 | 48 | 134 | 0.546 | 8.685 | 15.659 | 25.510 |
| Stored | | 28823 | 28823 | 2221 | 721 | 18 | 25 | 0.094 | 1.590 | 2.494 | 3.852 |
| Overflow | 6367 | | 76367 | 13159 | 3072 | 45 | 128 | 0.521 | 8.296 | 14.966 | 24.377 |
| Stored | | 32823 | 32823 | 2835 | 867 | 21 | 31 | 0.118 | 1.978 | 3.188 | 4.984 |
| Overflow | 6368 | | 64368 | 11069 | 2587 | 38 | 108 | 0.438 | 7.004 | 12.630 | 20.514 |
| Stored | | 44823 | 44823 | 4925 | 1352 | 28 | 52 | 0.201 | 3.270 | 5.524 | 8.848 |
| Overflow | 6369 | | 60368 | 10340 | 2419 | 36 | 100 | 0.410 | 6.551 | 11.807 | 19.163 |
| Stored | | 48823 | 48823 | 5654 | 1520 | 30 | 59 | 0.230 | 3.724 | 6.347 | 10.198 |

Notes: (1) 1983 sewer system.
(2) Tank size in m3.